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7.

THE BENEFICIAL USE
OF
STORM WATER
FINAL REPORT

"A SYSTEM STUDY, DESIGN, AND EVALUATION OF THE LOCAL STORAGE, TREATMENT, AND REUSE OF STORM WATER"

HIT-344

August 1968.

The research upon which this publication is based was performed pursuant to Contract No. 14-12-20 with the Federal Water Pollution Control Administration, Department of the Interior

HITTMAN ASSOCIATES, INC.
COLUMBIA, MARYLAND

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This final report covers the work performed by Hittman Associates, Inc. in cooperation with The Rouse Company and the "new city" of Columbia, Maryland, on the collection, storage, treatment, and reuse of storm water as a means of pollution control and water supply augmentation. ~~This work was performed for~~ the ~~Federal Water Pollution Control Administration, U. S. Department of the Interior, under Contract No. 14-12-20.~~

~~This report contains a~~ A discussion is presented of the work performed in the analysis and optimization of the system; the conceptual designs of potable, sub-potable, and pollution control systems; the conceptual design of a system for controlling pollution using a "conventional design" approach, the results of the system evaluation and plans for the demonstration of the local storage, treatment, and reuse of storm water. The appendices to this report include the derivations of the hydrology equations, survey reports on water usage and the public acceptance and legal aspects of reuse, design reports on the "conventional" and demonstration system, detailed estimates on the cost of the demonstration program, and descriptive information on packaged water treatment plants.

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FOREWORD

This final report covers the work performed by Hittman Associates, Inc. in cooperation with The Rouse Company and the "new city" of Columbia, Maryland, on the collection, storage, treatment, and reuse of storm water as a means of pollution control and water supply augmentation. This work was performed for the Federal Water Pollution Control Administration, U.S. Department of the Interior, under Contract No. 14-12-20.

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Storm water runoff from local areas is normally considered to be a form of wastewater. Use of storm water from surface water supplies normally occurs only after the runoff has undergone natural treatment processes and becomes a concentrated source. In this report, the local utilization of storm water runoff is referred to as "reuse" based on the normal classification as wastewater and to distinguish the type of usage from the normal use of surface water supplies.

I. SUMMARY AND INTRODUCTION

Simply stated, the local storage, treatment, and reuse of storm water is a concept for the control of storm water pollution in which the storm water runoff is collected in small storage basins dispersed throughout an urban area, treated to remove pollutants, and further treated for use. With this concept, the benefits derived from the use of storm water are used to offset the cost of effecting pollution control of storm water. This final report on "The Beneficial Use of Storm Water" contains a description of the system study, design, and evaluation of the local storage, treatment, and reuse of storm water and the analysis of the economic and technical feasibility of this concept.

The system study is based on the new city of Columbia, Maryland, and specifically Wilde Lake and its associated watershed which is located in the first village of Columbia. Wilde Lake is an artificial lake constructed in 1966, surrounded by apartments, townhouses, and individual residences, now approaching completion. This study is further based on the land development plans for this area, established lot boundaries, and other physical, aesthetic, and sociologic requirements existing within the area. The objective of this study has been to develop systems that could control the storm water pollution of Wilde Lake and to perform a comparative evaluation of the performance and economic aspects of these systems.

As an initial effort, it was necessary to develop quality standards for the reuse of storm water and to estimate the water requirements within the area for each quality classification. Four classes of water usage and associated quality standards were defined. These are Class "AA", "A", "B", and "C" which can be generally categorized respectively as: drinking and cooking water; laundry and personal external uses; lawn watering; and toilet flushing. Within the Wilde Lake watershed the demands for water in each of these classes are 123,700; 380,970; 282,760; and 320,460 gallons per day, respectively, with a total mean daily water demand of 1,107,560 gallons per day when the area is fully developed.

As a second initial effort, it was necessary to determine the quantities of storm water runoff that would have to be collected and the amounts of water that would be available for reuse. Since no gaging data was available for the Columbia area and since the runoff would be appreciably changed by the development of the area, methods were developed for producing long term synthetic hydrographs of storm water runoff as a function of rainfall and watershed characteristics. This was accomplished by regression analysis techniques using data from a gaging station located about 20 miles away in an area similar to Columbia. The model finally developed uses a third order polynomial to predict runoff as a function of rainfall and coefficients that can be modified to account for the degree of imperviousness of the developed area. In addition to predicting the overall quantities of water that would be collected, it was also necessary to predict peak flows and volumes associated with storms of various return intervals. A linear storage model with delay intervals was used in this analysis. As a result of this work, which is further described in Chapters II, III, and IV of the report, the annual mean runoff of the Wilde Lake watershed was calculated to be nearly 247 million gallons and the runoff volume from storms of a one year return interval were used to size the storage reservoir and treatment facilities.

With the estimates of water requirements as a function of quality and with methods for predicting the amount of runoff water that would be available, the problem became one of selecting the best sites for the location of the storage reservoirs and treatment plants. Three types of systems for local collection, storage, treatment, and reuse of storm water were considered: collection, storage, and treatment for potable reuse; collection, storage, and treatment for sub-potable reuse; and collection, storage, and treatment for pollution control. With three types of systems, four classes of water, and the numerous alternatives with respect to construction methods and locations for the individual facilities, the problem of selecting the best designs took on major proportions.

It was recognized from the initiation of the project that the local storage system would have to be optimized to the maximum possible extent for such a system to be economically attractive. In general, the unit costs of water storage and treatment are reduced as the size of facilities and process quantities are increased. Accordingly, the system analysis was directed toward the determination of the "least cost" system. A general model which would describe the various types of local storage and treatment systems was developed. A flow sheet and mass balance of the quantities of water that would enter and be processed through the system was prepared. A computer program was developed to perform the calculations of storage capacity, flow rates, etc., for each of the individual installations and for combinations of installations. Construction, operation, and maintenance costs for the various possible designs for each type of facility were prepared on a parametric basis and were used as inputs to the program. The system analysis computer program included provision for calculating the costs for each type, design, and location of installation and the benefits to be gained from the reuse of water. The least cost options were determined by the computer and printed on the output sheets. By the use of the system analysis program, it was possible to analyze over 100,000 possible system configurations. A detailed description of the system analysis program and the basis of its development is contained in Chapter II.

The main output of the system analysis program was a computer generated report in which the net benefits or costs for each possible system configuration were listed as a function of the reuse quality of water. From this listing, the possible combinations were ranked in order of maximum benefit or least cost. Topographic and land use maps of the area were then used to determine the maximum size reservoirs that would be possible or acceptable at each location. From the ordered listing of maximum benefits or least cost and the maximum possible reservoir sizes at each location, the optimum feasible system configuration for each type of local storage system were selected. Examples of the output sheets generated by the system analysis program with a discussion of how they were used in the selection of the optimal feasible systems are contained in Chapter III.

Conceptual designs were prepared for three types of systems of local collection, storage, treatment, and reuse of storm water. Each of the systems consists of 10 storage reservoirs and pretreatment units. Chapter IV contains descriptions of each of these systems, with a discussion of the performance characteristics and summarized the design, construction, operating and maintenance costs. The results of the conceptual design work are as follows:

- The "Potable Reuse System" provides pollution control at the least cost of the three local storage and treatment systems. Construction cost is estimated at \$1,445,000. The total daily cost of this system, including amortized capital cost and operating and maintenance costs, will be \$401. It will supply an average of 556,000 gallons of water per day, or 52.6 percent of the total demands in the watershed, worth \$250 for a net daily cost of \$151.
- The "Local Pollution Control System," without reuse, provides pollution control at a cost slightly higher than the Potable Reuse System. Construction of this system will cost an estimated \$830,000. The total cost of such a system will be \$174 per day.
- The "Sub-Potable Reuse System" is the most expensive of the three local storage and treatment schemes. This system will cost an estimated \$1,036,000 without a distribution system, and \$2,598,000 with a sub-potable water distribution system. The total daily cost of such a system will be \$300 per day not including the distribution system or \$524 per day including the distribution system. This system will supply an average of 460,000 gallons of water per day or 41.6 percent of the total demands of the watershed, worth \$207 for a net daily system cost of \$93 without distribution or \$317 with distribution.

In parallel with the system analysis and conceptual design of the local storage system, conceptual designs were prepared for a pollution control system using conventional design methods and available technology. This work was performed by Whitman, Requardt and Associates of Baltimore, Maryland. It was used to provide a basis for the comparison of the performance characteristics and technical and economic aspects of the local storage system. The "Conventional Treatment System" consists of a number of gravity interceptors, two pumping stations and associated force mains, a holding pond, and a treatment basin at the head of Wilde Lake. This system, like the local storage system, provides for the retention and treatment of a one year storm. The Conventional Treatment System, due to available space limitations, could not be designed to provide equivalent treatment with respect to suspended solids. The Conventional Treatment System is estimated at \$1,314,000 construction cost and total daily costs, including amortized capital costs and operation and maintenance costs of \$257 per day. The Conventional Treatment System is further described in Chapter V and Appendix D of this report.

The system evaluation phase of this project consisted of a detailed comparison of the performance characteristics and the technical and economic aspects of the four pollution control systems. This also included consideration of the secondary benefits that might accrue from the use of each of the systems. The system evaluation phase is discussed in Chapter VI. A comparison of the costs for the four pollution control systems which were considered is as follows:

	$/year	$/acre-year	$/D.U.* Year
Potable Reuse System	55,200	48.05	14.68
Local Pollution Control	63,400	55.25	16.88
Sub-Potable Reuse System			
Without Distribution Costs	34,000	32.57	9.05
With Distribution Costs	115,800	104.30	30.82
Conventional Treatment System	93,800	81.71	24.97

*Dwelling Unit

Based on this analysis, the local storage and treatment systems are economically more attractive than the Conventional Treatment System for both the potable reuse and local pollution control cases. The distribution costs for the Sub-Potable Reuse System make this system economically unattractive compared to the other three systems in the case of the Wilde Lake watershed. A Sub-Potable Reuse System might be attractive in an area where there is an appreciable demand for sub-potable water, i.e., a large industrial user which could be supplied with a low cost distribution system.

With respect to the performance characteristics, the local storage system provides a higher degree of pollution control than the Conventional Treatment System. In terms of secondary benefits, the local storage system will dampen the increased runoff flows due to urbanization to near natural levels and may substantially abate post-development stream bed erosion and the resultant pollution. Other secondary benefits are cited in Chapter VI.

As a part of this project, plans and schedules were developed for possible programs for the demonstration of the local storage, treatment, and reuse of storm water. Four possible approaches were considered and are presented in Chapter VII. Three of the approaches are based on the construction, operation, test, and evaluation of various facilities and systems to demonstrate the performance characteristics, technical and economic aspects, and effectiveness of prototype local storage, treatment, and reuse systems. The fourth approach envisions a program in which the pollution control and reuse aspects of the local storage and treatment system would be demonstrated on a limited scale. This would be followed by a demonstration of the complete system on the Wilde Lake watershed. Chapter VII also contains estimates of the costs associated with each of these approaches and a discussion of the extent to which each approach would attain the objectives established for the demonstration program.

As a part of the demonstration program planning, contacts were made with the developers of Columbia, the Columbia Parks and Recreation Association, Howard County Officials, and the State of Maryland, Department of Health and Department of Water Resources. Considerable interest was found in the local storage and treatment concept and these groups expressed a willingness to participate in the demonstration program. A summary of the comments from these groups is contained in Section VII. H.

The various approaches for the demonstration program were evaluated from the standpoint of cost, time, attainment of objectives, and disposition of facilities. Based on this evaluation, it was concluded that the best approach would be the construction of three reservoirs to demonstrate the system aspects of storm water pollution control, followed by the installation of a packaged water treatment plant to demonstrate the reuse of potable water. This program could be completed within a two year period and was estimated to have an overall cost of $752,400. This program could be phased. The increments of a phased program would consist of; two gaging and sampling stations, one storage reservoir and pretreatment unit, installation of a packaged treatment plant, two additional storage reservoirs and pretreatment units and a connection to the public water system. The two additional storage reservoirs could be eliminated to reduce the cost of the demonstration program to less than $500,000. Further discussions with local and State agencies to obtain interest in participating in a demonstration program are planned.

In summary, it was concluded that this system study has shown that the local storage, treatment, and reuse of storm water is a practical and economically attractive method of providing pollution control in urban areas. Although the designs of the systems for the collection, storage, and reuse of storm water incorporate concepts that have not been used in storm water pollution control, the available data and methods used in the analysis provide excellent assurance that such systems will perform as expected. It is believed that with a rather modest demonstration program, the local storage and treatment will be adequately proven to permit this concept to be applied directly to the solution of future storm water and combined sewer pollution problems. Further, it is believed that such a demonstration program can materially contribute to the general technology in the urban hydrology field.

II. SYSTEMS ANALYSIS

A. Introduction

In order to subject the local storage, treatment and reuse of storm water concept to a comprehensive examination, taking full account of interactions between the various components and their environment, the systems analysis technique was employed. Since most system elements are substantially affected, if not determined, by the physical environment in which they are to be used, a completely generalized study was not attempted. Instead, a specific area was chosen where the study objective could be easily defined, the environmental constraints established and quantified, and where the conclusions of the systems study could be verified by physical demonstration. The specific criteria used in selecting the study area are fully described in Section II.B, along with a physical description of the selected area.

The systems analysis effort itself was conducted in the following steps:

(1) Definition of the system
(2) Formulation of the objective function
(3) Identification of the constraints
(4) Investigation of the constraints
(5) Maximization of the objective function subject to the constraints

Figure II-1 illustrates, diagramatically, the complete water system within a specific watershed. Rainfall and water from a public supply are inputs and evapo-transpiration, soil percolation, surface drainage out of the watershed, consumptive uses, and sanitary sewer flow are outputs. The parts of this overall system being subjected to the present analysis are:

(1) Storm Water Storage
(2) Storm Water Pretreatment
(3) Final Treatment
(4) Treated Water Storage
(5) Treated Water Uses

The first four elements can be varied in size and performance within physical and technological constraints and the last consists of some fraction of the total water demand, this total being fixed for a given watershed. These elements are shown in simplified form in Figure II-2, the Storm Water Storage, Storm Water Pretreatment, Final Treatment, and Treated Water Storage elements being collectively described as Local Storage, Treatment, and Reuse of Storm Water.

Formulation of the objective function consists of determining both the objectives of the system and a method for measuring the relative achievement of these objectives. Pollution abatement problems present serious

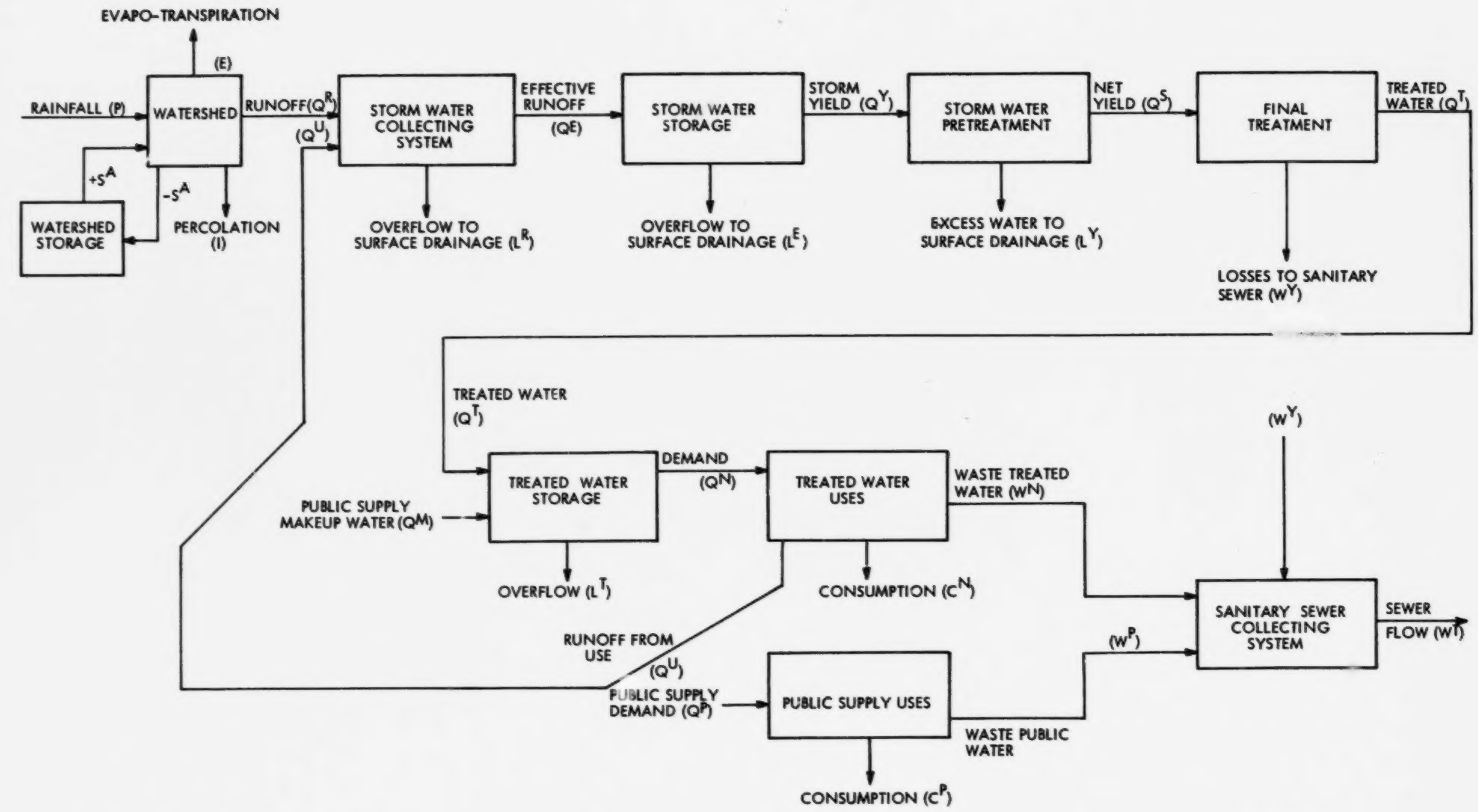
EVAPO-TRANSPIRATION
(E)
RAINFALL (P)
WATERSHED
RUNOFF(Q^R)
(Q^U)
+S^A
-S^A
WATERSHED STORAGE
PERCOLATION (I)
STORM WATER COLLECTING SYSTEM
OVERFLOW TO SURFACE DRAINAGE (L^R)
EFFECTIVE RUNOFF
(QE)
STORM WATER STORAGE
OVERFLOW TO SURFACE DRAINAGE (L^E)
STORM YIELD (Q^Y)
STORM WATER PRETREATMENT
EXCESS WATER TO SURFACE DRAINAGE (L^Y)
NET YIELD (Q^S)
FINAL TREATMENT
LOSSES TO SANITARY SEWER (W^Y)
TREATED WATER (Q^T)
TREATED WATER (Q^T)
PUBLIC SUPPLY MAKEUP WATER (Q^M)
TREATED WATER STORAGE
OVERFLOW (L^T)
DEMAND
(Q^N)
TREATED WATER USES
CONSUMPTION (C^N)
WASTE TREATED WATER (W^N)
RUNOFF FROM USE
(Q^U)
PUBLIC SUPPLY DEMAND (Q^P)
PUBLIC SUPPLY USES
CONSUMPTION (C^P)
WASTE PUBLIC WATER
(W^P)
(W^Y)
SANITARY SEWER COLLECTING SYSTEM
SEWER FLOW (W^T)

Figure II-1. Complete Water System Incorporating Local Storage, Treatment and Reuse of Storm Water

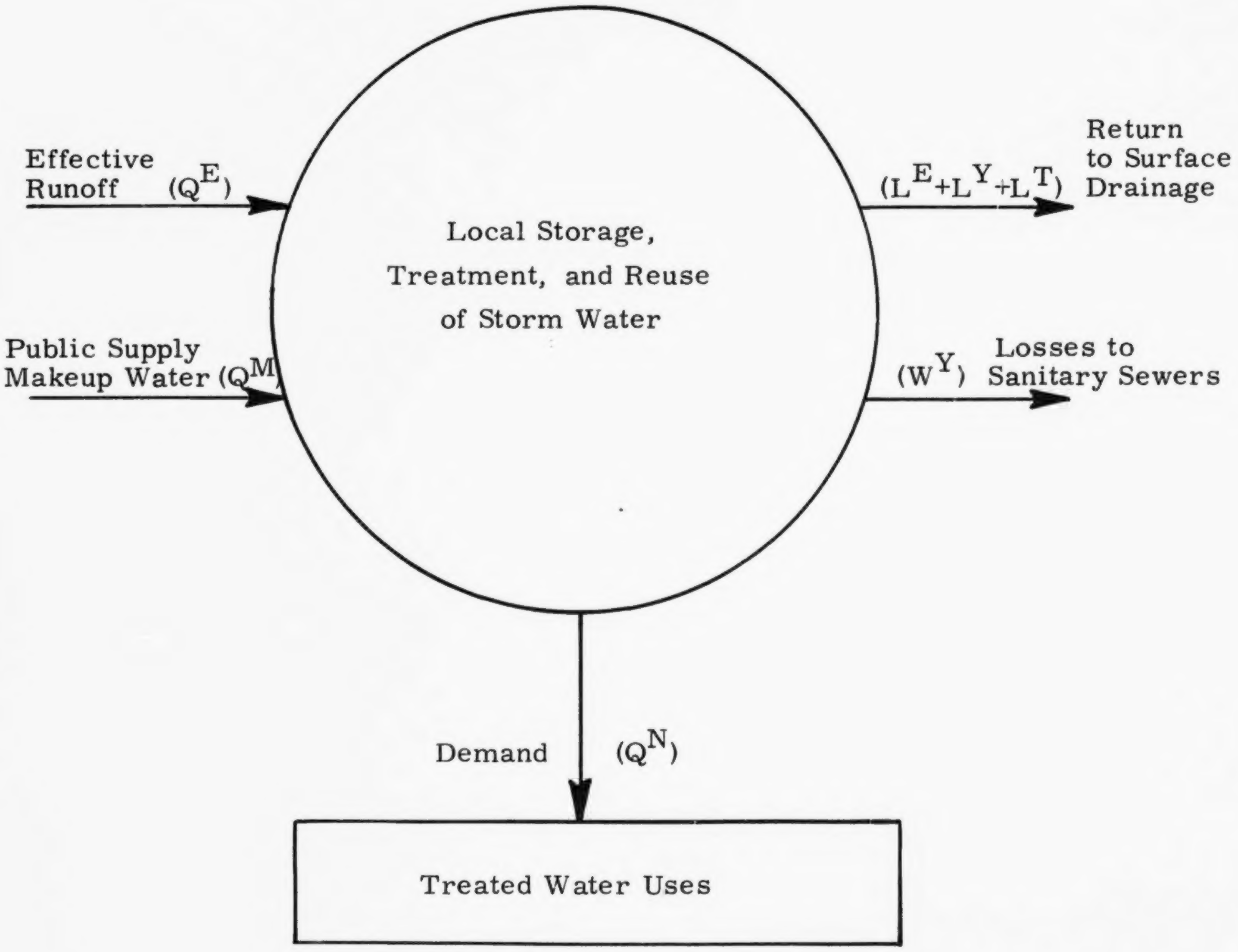
Effective
Runoff (Q^E)
Public Supply
Makeup Water (Q^M)
Local Storage,
Treatment, and Reuse
of Storm Water
(L^E+L^Y+L^T)
Return
to Surface
Drainage
(W^Y)
Losses to
Sanitary Sewers
Demand
(Q^N)
Treated Water Uses

Figure II-2. Diagram of Study Concept

difficulties in the construction of precise, quantitative objective functions, due to the lack of a clear, generalized definition of pollution itself. A common approach is to specify effluent standards and to require all water entering the protected area to meet certain tests. These tests and the corresponding limits must be selected by a combination of objective and subjective processes, tempered by the availability of convenient test procedures. This analysis assumed that effluent standards could be set for the study site which would achieve the desired pollution abatement objective.

The objective function is described as follows:

(1) Water released to surface drainage from the Local Storage, Treatment, and Reuse of Storm Water system shall meet or exceed stated effluent standards, and;

(2) This condition will be achieved at the lowest net system cost.

This function is subject to all applicable physical, technological, legal, and institutional constraints, as described in the following sections. The term "lowest net system cost' is understood to be interchangeable with the term "highest net system benefit" and is defined as

$$\min\left(\Sigma C_a + a\,\Sigma C_f - \Sigma B_a\right) \qquad \text{(II-1)}$$

or, stated differently

$$\max\left(\Sigma B_a - \Sigma C_a - a\,\Sigma C_f\right) \qquad \text{(II-2)}$$

Where

C_a = marginal annual operating and maintenance costs
C_f = marginal fixed construction and project costs
B_a = marginal annual benefits
a = capital recovery factor*

Equation II-2 is described as the "net benefit function" and subsequent discussion will be in terms of maximizing this function.

Section II B discusses criteria employed in the selection of a study site. One of these guidelines was a requirement that the study include the entire watershed of a specific body of water. This permits effluent standards to be set for released water that effectively determine the water quality in the body of water to be protected. In the case of the area selected, effluent standards were made equivalent with the desired water quality in the receiving body. These standards are discussed in detail in Section II. E. 3.

Sections II. C through II. H itemize and describe the physical and technological constraints identified as well as certain legal and institu-

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*$a = \dfrac{i}{1-\dfrac{1}{(1+i)^m}}$ where: i = annual rate of interest, or cost of capital
m = period of amortization, in years

tional constraints. Further discussion of these last two categories can be found in Section VI.F and Appendix F. The maximum of the objective function and the resulting optimum conceptual systems are discussed in Chapters III and IV.F.

B. Site Selection

1. Columbia Area

Early in the formulation of the Storm Water Reuse program, it became apparent that the new city of Columbia, Maryland, offered several unique advantages as a study site for the systems analysis. The Storm Water Reuse concept is specifically directed to urbanized areas where water runoff can be clearly identified as a source of water pollution. In order to perform a systems analysis in such an area, the physical characteristics of the area must be known in some detail, including hydrologic factors, land uses, population densities, etc. Since the method of analysis was to be kept generalized wherever feasible, a study area was deliberately selected where gage data was not available for rainfall and runoff. The requirements that led to the close examination of Columbia, however, were those related to the planned demonstration of the concept following analysis.

Columbia is a completely planned new town being developed by Howard Research and Development Corporation of Baltimore, Maryland, an affiliate of the Rouse Company, on a 27 square mile site in Howard County, Maryland (Figure II-3). Designed for a 1980 population of 110,000 people, Columbia is midway between Baltimore and Washington astride the busy Northeast Corridor. Construction started in 1966 and the first residents moved into the city in June, 1967. By the first anniversary of the arrival of these residents, more than 5000 people were living in the community and construction was proceeding on schedule. The complete town will consist of nine villages, each village made up of a number of neighborhoods. A principal feature of the Columbia plan is the extensive use of water as a focal point for the activities of the community. The town site includes portions of the Little Patuxent, Middle Patuxent, and Patuxent river basins and five man-made lakes totaling over 500 acres of water surface are scheduled. Two of these, Lake Kittamaqundi and Wilde Lake are already in existence.

The availability of the complete Columbia plan permits the conduct of a systems study of the Storm Water Reuse concept in an area which is not yet fully developed. If this study is based on the planned characteristics of the area after development, a demonstration program can be conducted which will fully evaluate the results of the systems analysis while taking advantage of the reduced costs of concurrent construction. Since the entire area is being developed by one organization, coordination and cooperation are greatly enhanced. Another fortunate feature of Columbia is the integration of many different types of land use, including low to high density residential development, numerous types of commercial uses, and various industrial uses. In some areas of Columbia, a wide variety of these uses can be found in a single watershed. As a result the influence of various land uses on storm water quality and quantities as well as the accompanying possibilities for reuse can be evaluated in the systems study and subsequent demonstration.

2. Wilde Lake Watershed

In order to simplify hydrologic aspects of the systems study as well as to create an easily definable water quality objective, it was decided to

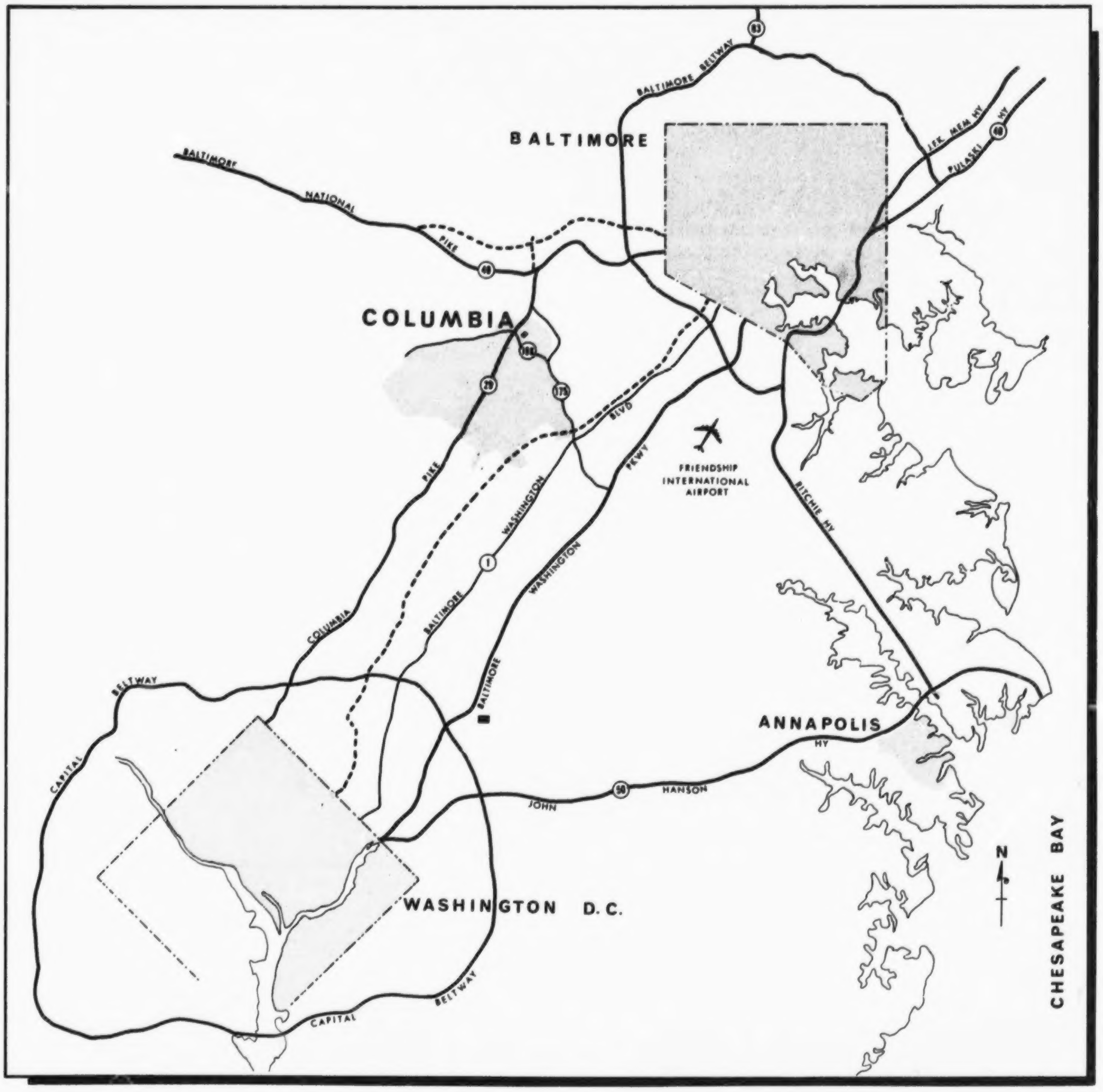

83
BALTIMORE BELTWAY
BALTIMORE
J.F.K. MEM HY
PULASKI HY
40
BALTIMORE NATIONAL PIKE
40
COLUMBIA
108
29
175
BLVD
PKWY
FRIENDSHIP INTERNATIONAL AIRPORT
PIKE
WASHINGTON
1
BALTIMORE
WASHINGTON
RITCHIE HY
COLUMBIA
BALTIMORE
CAPITAL BELTWAY
ANNAPOLIS
HY
50
JOHN
HANSON
N
CHESAPEAKE BAY
WASHINGTON D.C.
CAPITAL BELTWAY

Figure II-3. Location Map of Columbia, Maryland

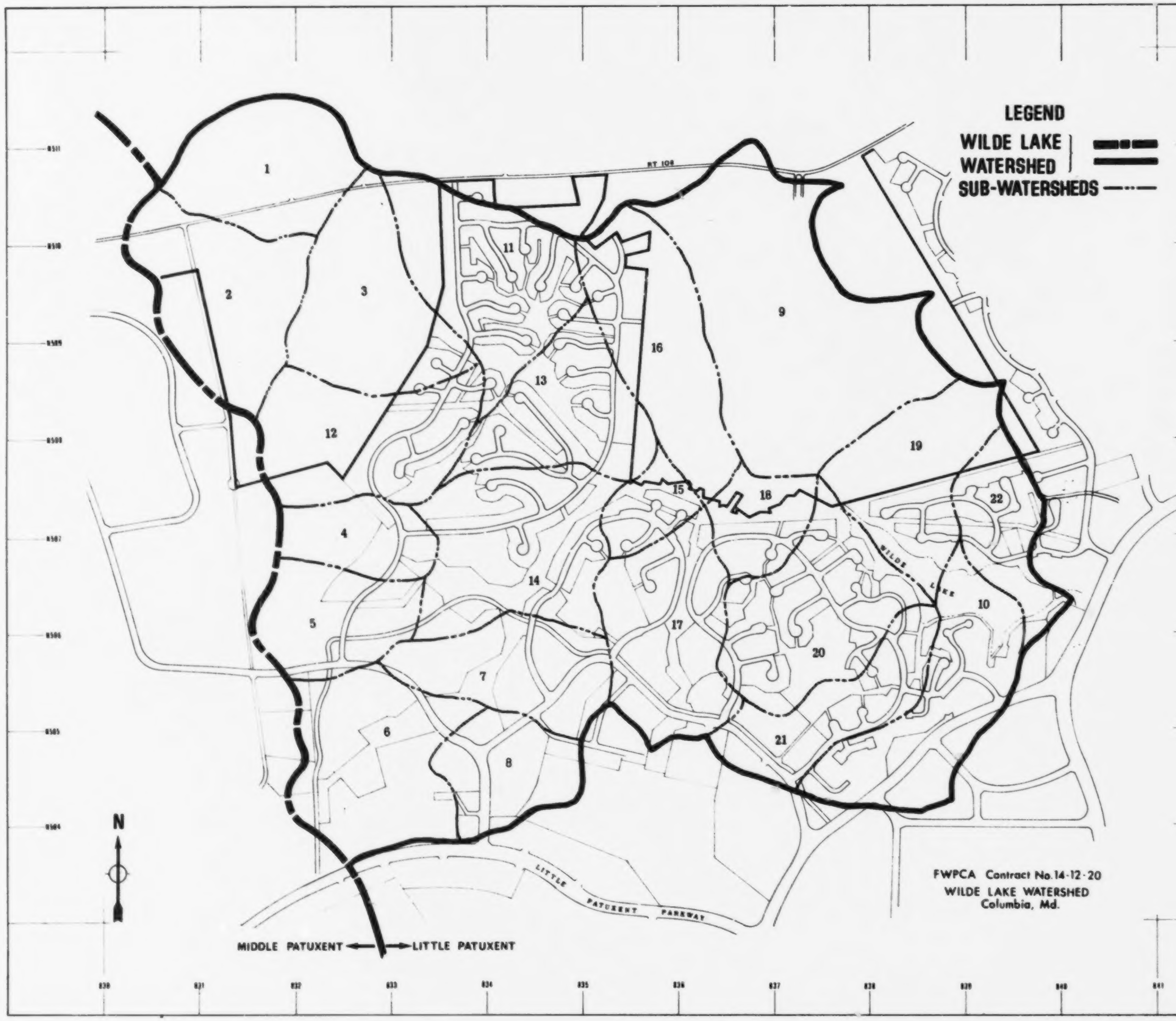
perform the study on the drainage area of one of the artificial lakes. Of the two lakes which would be available for the demonstration program, only one, Wilde Lake, had a drainage area substantially within the Columbia project. Furthermore, a great diversity of land uses exists in the Wilde Lake watershed. Construction in the watershed is scheduled for completion before 1970, permitting concurrent construction of demonstration facilities, followed almost immediately by operation of these facilities in a fully developed area.

The Wilde Lake watershed consists of 1140 acres, or about 1.8 square miles. Other information concerning the lake and its watershed is listed in Table II-1. Figure II-4 is a map of the area, showing a number of sub-watersheds which were used in the hydrologic analysis reported in Section II. C.

TABLE II-1. THE WILDE LAKE WATERSHED, COLUMBIA, MARYLAND

Watershed Area	1,140	Acres
Planned Population	12,254	Persons
No. Dwelling Units	3,757	
Planned Land Use - Columbia		
Open Space	147.6	Acres
Single Family Res. - Low Density	13.3	"
Single Family Res. - Medium Density	185.8	"
Town-houses	114.8	"
Garden Apartments	66.6	"
Mid-rise Apartments	4.9	"
Employment Centers	24.8	"
School Sites	37.7	"
Public Rights-of-way	115.8	"
Total - Columbia	711.3	"
Expected Land Use - Outside Columbia		
Open Space	194.5	Acres
Single Family Res. - Low Density	191.0	"
Public Rights-of-way	22.1	"
Total - Outside Columbia	407.6	"
Wilde Lake Water Surface Area	21.1	"
Estimated Maximum Capacity	48,200,000	Gallons
Fraction Impervious Construction in Watershed	0.22	

With the cooperation and assistance of Howard Research and Development Corporation, the Wilde Lake watershed was chosen as the site for the systems analysis effort of the Storm Water Reuse program, and the southwest quadrant of the watershed was identified as the tentative site of a first phase demonstration facility following satisfactory completion of the systems analysis. To permit evaluation of various storm water reuse concepts in as many configurations as possible, the study watershed was divided into 22 sub-watersheds. These were delineated by natural ridge lines wherever possible, permitting the runoff from each area to be collected at a single point.


LEGEND
WILDE LAKE WATERSHED
SUB-WATERSHEDS
RT 108
WILDE LAKE
LITTLE PATUXENT PARKWAY
N
MIDDLE PATUXENT
LITTLE PATUXENT
FWPCA Contract No. 14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
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22
8511
8510
8509
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841

Figure II-4. Wilde Lake Watershed - Sub-Watershed Map

C. Storm Water Hydrology

1. Runoff Data Requirements

As one of the initial phases of the Storm Water Reuse project, it was necessary to develop methods of adequately describing the storm water flow characteristics of the Wilde Lake drainage area. Two different types of hydrologic data were required:

(a) A continuous daily runoff hydrograph from each of the sub-watersheds in the basin.

(b) Peak flows in each sub-watershed as a function of rainfall recurrence interval. .

The first of these was needed to estimate the total volume of water available for reservoir storage and reuse for any time interval, and the second to estimate sedimentation efficiency and system operation during large storms. Methods were required for obtaining both types of data without benefit of gaged stream flow records at Wilde Lake.

Fortunately, many methods exist for calculating peak flows from drainage areas lacking stream flow data. There are, however, few methods which can be applied to the development of a daily runoff hydrograph in an area where basic hydrologic data are unavailable. Since this problem was the first to be approached and required considerable effort for solution, it will be discussed in detail in the following sections.

2. Characteristics of Available Methods for Calculating Continuous Runoff Hydrographs

The method to be used in the derivation of a runoff hydrograph for the sub-watersheds of Wilde Lake had to satisfy the following requirements:

(a) It had to provide a continuous hydrograph of daily runoff volumes for each sub-watershed and be applicable to the variety of impervious conditions of the watersheds.

(b) It had to be a fairly simple method requiring a minimum of data input and a minimum amount of time spent in its development.

(c) It had to be derivable without any basic hydrologic data except land use and rainfall.

For small rural watersheds such as Wilde Lake, there exists no standard simple method of providing a runoff model that would satisfy these requirements. To develop such a method would have required more effort than could have been justified under this project since the selection of runoff values satisfied only a small part of the project goals. Therefore, several existing approaches to the solution of similar problems were examined with the purpose of selecting the best suited approach and modifying it if necessary.

Each of the methods that were examined has been used in the past to estimate runoff volumes and uses as one of the input parameters a rainfall record for the drainage area. The following approaches were considered.

a. Unit Hydrograph. This involves calculating for a drainage area the hydrograph that would be produced by one unit of "effective precipitation" (actual rainfall minus inflitration and other losses) occurring during a specified period of time (Ref. 1). This unit hydrograph is then used to calculate the runoff hydrograph from any distribution of effective rainfall. There are two disadvantages to this approach. On small drainage basins, the time intervals which would be required for rainfall input would be measured in minutes, requiring a great volume of data. In addition, the calculation of effective precipitation from actual precipitation would require a knowledge of infiltration losses during each rainfall at any time of the year, and this information does not exist for ungaged drainage areas.

b. Linear Storage Reservoir Models. This computer method, which has been employed by the U.S. Department of Agriculture (Ref. 2), utilizes routings of effective precipitation through linear storage reservoirs with a built-in lag time. This method gives excellent results for individual storms, but also requires short-time interval inputs as well as infiltration knowledge to calculate effective precipitation.

c. Digital Simulation Models. Several models of this type exist with many variations. The basic problem with them, however, is that they require a great deal of detailed input information to describe the watershed. For example, the Stanford Hydrologic Model gives the watershed water balance (surface runoff, storage, and ground water flow) at any time, but requires evapotranspiration information, arrays describing basin interflow and infiltration characteristics, and 32 constants describing the physical characteristics of the area. Although these models generally describe accurately an area where there is gaged runoff information to assist in choosing the constants, it is doubtful that they are nearly as accurate where gaged runoff information does not exist.

d. Regression Models. This method takes a dependent variable and one or more independent variables and derives the coefficients relating the variables so that the variation between the observed dependent variable and the calculated dependent variable is at a minimum. Regression equations involving rainfall and runoff have been used in past hydrologic studies in areas where gaged data exist. However, it is possible to derive a regression equation for a gaged watershed and apply it to another ungaged area as long as the areas are physically and hydrologically similar.

It was decided that the adoption of some form of regression model would probably be the most fruitful approach. The procedure that was chosen was to select from the Maryland area another watershed that had the same physical and hydrologic conditions as the Wilde Lake area and had at least five years of rainfall and runoff data. A regression analysis could then be performed on this watershed and the results applied to Wilde Lake.

The advantage of this appraoch is that a detailed knowledge of infiltration and soil conditions is not required since the overall effects of such parameters are indicated by the regression coefficients. The regression model does not have to present an exact cause-effect relationship between variables as long as its predictive ability is good and the variable relationship is reasonable.

3. Development of Regression Models

After examining the available runoff data in the Maryland area, only one watershed was located having physical characteristics similar to those of Wilde Lake. This watershed, on the West Branch of Herring Run in Baltimore County, is of the same approximate size and shape, has the same topographic features, and is located in the same geologic zone as the Wilde Lake drainage area. For these reasons it is ideally suited for the derivation of a rainfall-runoff model which could then be applied to Wilde Lake with modifications to account for varying percentage of imperviousness in the sub-watersheds.

The initial regression model chosen for the Herring Run data was a strictly linear one involving antecedent precipitation:

$$Q_i = a + b R_i + c(R_{i-1} + R_{i-2}) + d(R_{i-3} + R_{i-4} + R_{i-5}) \quad \text{(II-3)}$$

where:

Q_i = total runoff/square mile on i^{th} day

R_i = total rainfall on the i^{th} day

a, b, c, d = regression coefficients to be calculated

It was believed that the coefficient "a" would represent average base flow for the period of analysis and could be subtracted from the equation if only storm runoff effects were to be considered. The coefficient "b" represents the effects of precipitation on runoff during the same day, while "c" and "d" are measures of the effects of antecedent precipitation on runoff.

The procedure that was used was to take as input to the regression equation five years of daily streamflow, in cfs-days/mi^2 - day, as gaged by the U.S. Geological Survey, and the corresponding five-year period of daily rainfall in inches/day from the Towson, Maryland weather station, which is located less than two miles from the Herring Run drainage area. The annual precipitation at Towson for the period covered by the data averaged 39.79 inches, compared to the long-term average of 44.21 inches. These five years included one with above average rainfall and four with below average rainfall.

A computer program was used to solve for the regression coefficient in three different approaches:

(a) One equation for the drainage area representing one year, but based on the entire five-year data period

(b) Four equations, each representing a single season (with five years of data for each season)

(c) Twelve equations, each representing one month (with five years of data for each month)

After the initial computer run, it could be seen that the regression equations gave very low correlation coefficients. The square of the multiple correlation coefficient "R" represents the fraction of total variance between the observed and calculated values that is explained by the regression model. A close analysis of the input data indicated that the Towson weather station data is collected at 6:00 p.m. each day and that an evening thunderstorm would be reported as having occurred on the following day. This displacement in record could therefore indicate high streamflow on one day, with no indicated precipitation until the next day. By examining every instance of this type of reversal in the data and correcting the rainfall record by examination of hourly precipitation records from Friendship Airport, a total of 10 percent of the data points were altered. It was expected that this would have a great effect on the correlation coefficients in succeeding runs.

The first analysis also indicated that two other changes were desirable. The base flow term in the equation was altered to the form $A + B \sin(2\pi t/365)$ since it was noted by observation that base flow appeared to follow a sinusoidal pattern, high in the Spring and low in the Fall. The antecedent precipitation terms were also changed, since the regression coefficients indicated that the effects of antecedent precipitation more than two days past were negligible. Therefore, the second form of the equation became:

$$Q_i = a + b \sin\left(\frac{2\pi t}{365}\right) + cR_i + dR_{i-1} + eR_{i-2} \qquad \text{(II-4)}$$

where t = day of the year (1 to 365)

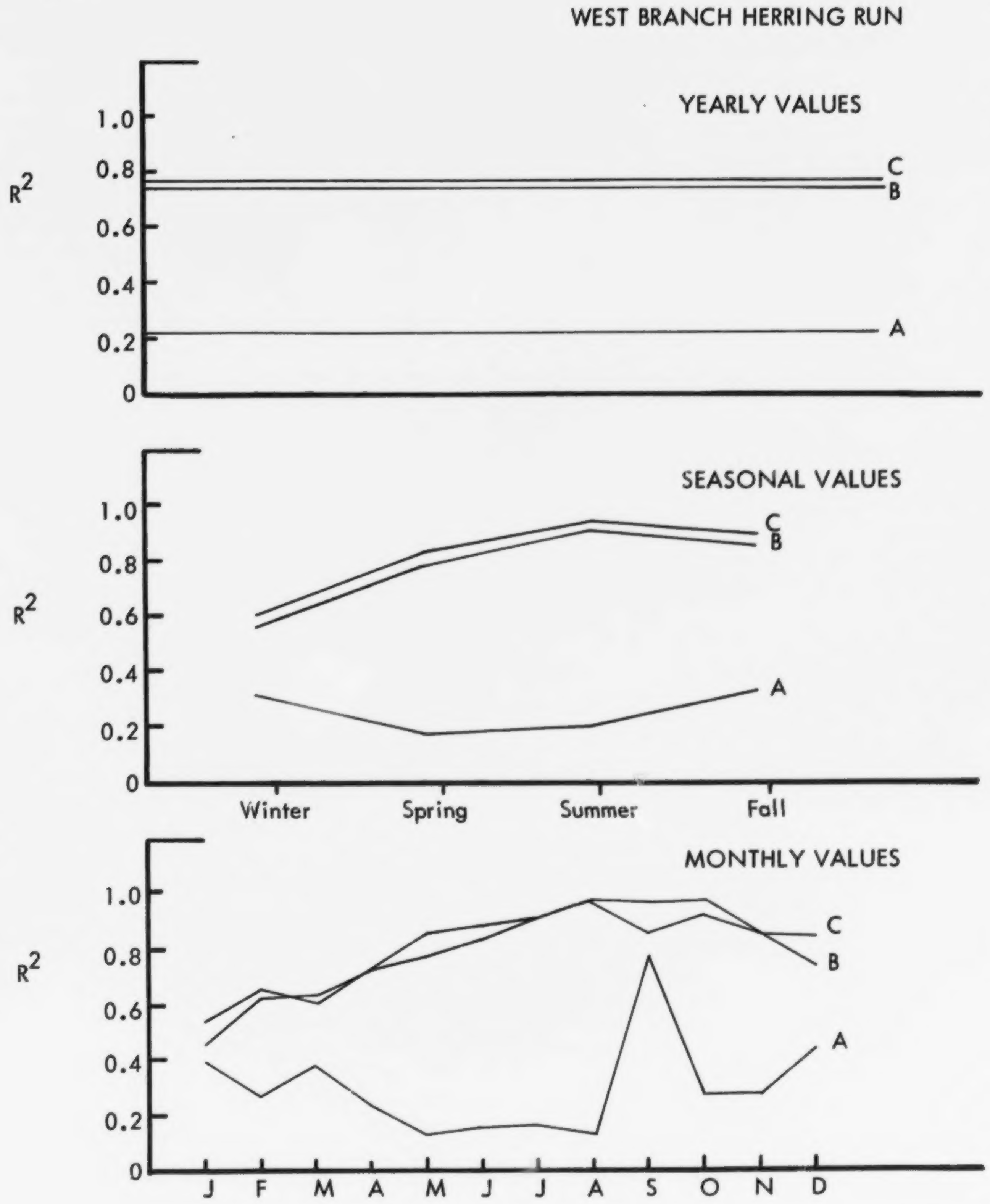
The results of correcting the data and altering the form of the equation were apparent in the improvement in correlation coefficients of the new equations (Figure II-5). The calculated base flow terms, when plotted, definitely indicated a sinusoidal variation throughout the year (Figure II-6)

After plotting many values of calculated runoff and observed runoff, it could be seen that for peak periods of rainfall (over 1.5 inches/day), the regression model consistently underestimated the volume of runoff. For this reason, another equation was chosen for analysis:

$$Q_i = a + b \sin\left(\frac{2\pi t}{365}\right) + cR_i + dR_i^2 + eR_i^3 \qquad \text{(II-5)}$$

This nonlinear equation could fit the lower values of rainfall as well as the other equation, but has the advantage of being able to give greater weight to the larger values of R. No antecedent precipitation terms were included in this equation since the coefficients from the previous analysis indicated very little effect due to antecedent rainfall.

After deriving the coefficients for this equation, the results were compared to the other runs. Figure II-7, another plot of correlation coefficients, shows a slight improvement over the previous trial. The largest effects were seen in actual runoff plots, where the polynomal equation predicted flows from large rainfalls much better than the linear equation.



R = MULTIPLE CORRELATION COEFFICIENT

R^2 for $Q_i = a + bR_i + c(R_{i-1} + R_{i-2}) + d(R_{i-3} + R_{i-4} + R_{i-5}) =$ ——— A

R^2 for $Q_i = a + b \sin(\frac{2\pi t}{365}) + cR_i + dR_{i-1} + eR_{i-2} =$ ——— B

R^2 for $Q_i = a + b \sin(\frac{2\pi t}{365}) + cR_i + dR_i^2 + eR_i^3 =$ ——— C

Figure II-5. Correlograms of Various Multiple Regression Equations - West Branch Herring Run

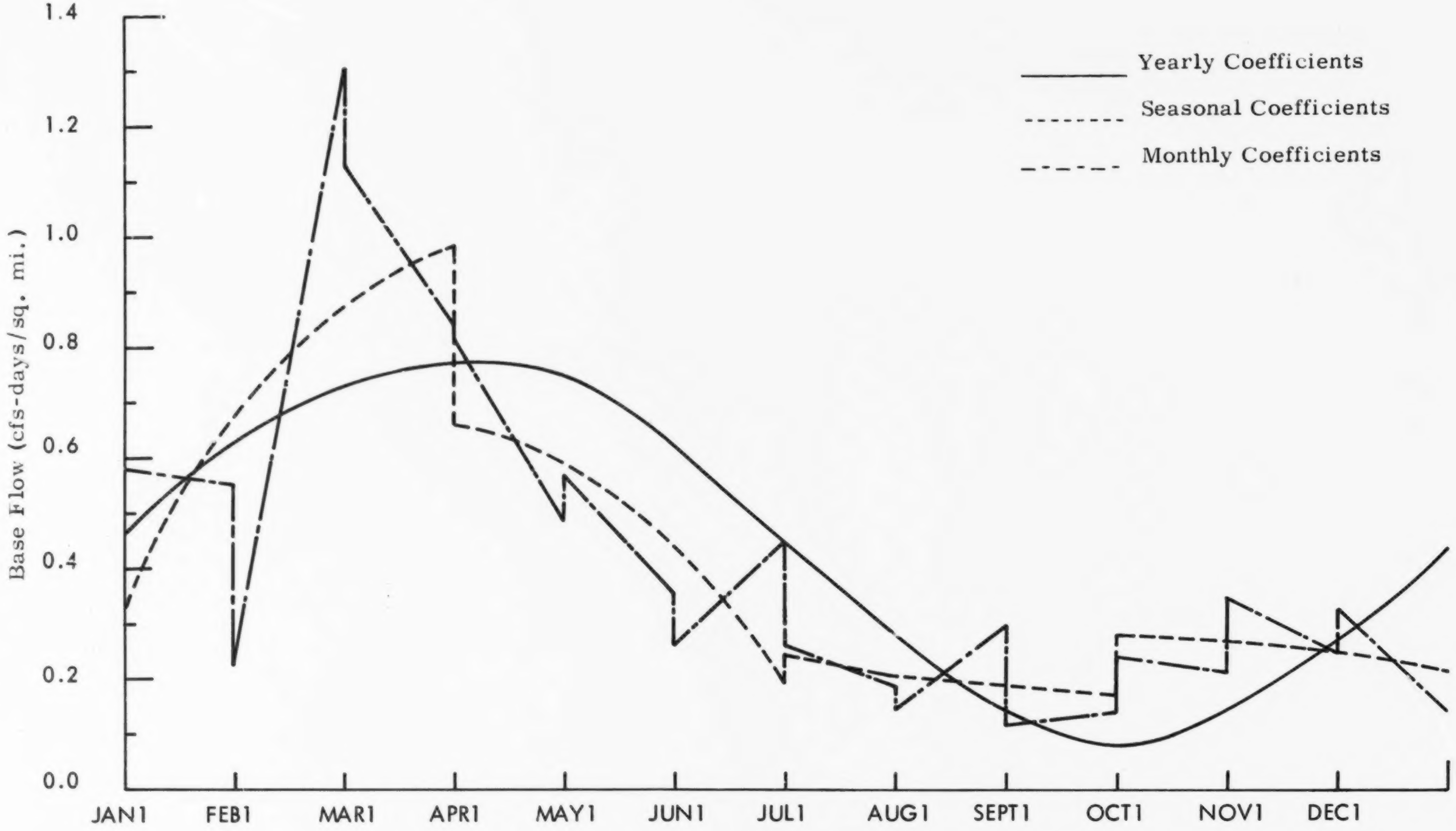


Figure II-6. Calculated Base Flows - West Branch Herring Run

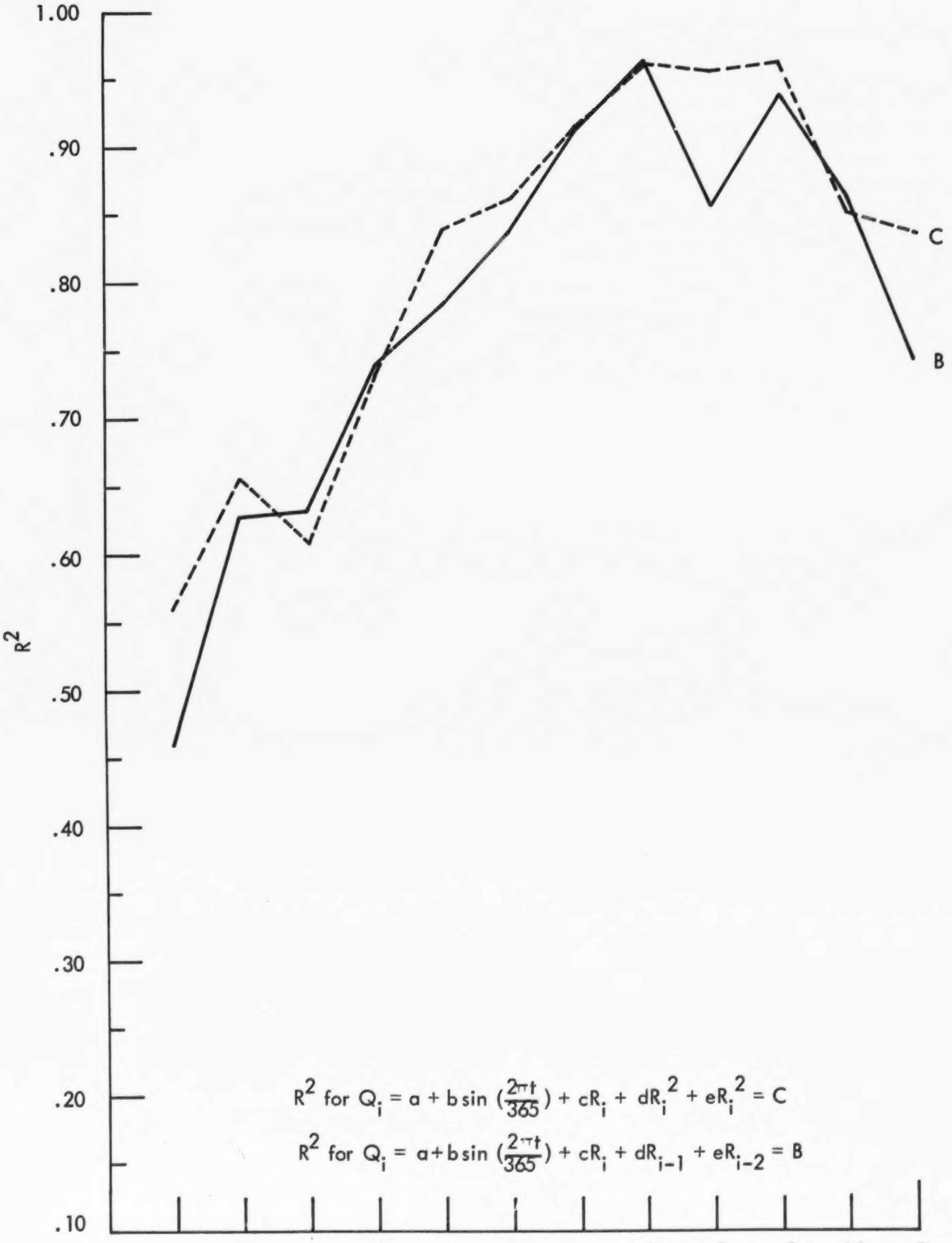


Figure II-7. Correlograms of Multiple Regression Equations
West Branch Herring Run

One final modification of the model was made. Although the sinusoidal base flow term has significance when dealing with the yearly equation, it loses its importance in the seasonal and monthly equations. Therefore, the model was run with only the constant portion of the base flow term since it had been decided to work with the seasonal equations. The final form of the equation, along with the derived regression coefficients and correlation coefficients, is presented in Table II-2.

TABLE II-2. FINAL REGRESSION EQUATION COEFFICIENTS

$$Q_i = A + BR_i + CR_i^2 + DR_i^3 \quad \text{(II-6)}$$

	Regression Coefficients				Correlation	
	A	B	C	D	R	R^2
Yearly	0.5247	3.016	1.187	-0.168	0.858	0.736
Seasonal						
Winter	0.885	3.449	0.808	0.099	0.756	0.572
Spring	0.625	2.044	1.917	-0.419	0.874	0.764
Summer	0.269	1.849	1.969	-0.260	0.974	0.949
Fall	0.353	1.297	4.591	-1.389	0.927	0.859

R_i = rainfall in inches/day

Q_i = runoff in cfs-days/mile^2-day

4. Modification of the Regression Model for Application to Wilde Lake

Although the Herring Run and Wilde Lake areas are similar in most respects, the sub-watersheds of Wilde Lake will have a variety of impervious percentages after development. Past studies of the effects of urbanization on runoff (Ref. 3) have shown that an increase in impervious cover will decrease the lag time between peak rainfall and runoff, increase the peak runoff rate, and increase the runoff yield per unit area. It is only this last effect which will affect the derived runoff model in this case, since the areas are too small for lag time changes to be important on a daily flow basis, and the peak flows will only affect the pipe sizings.

It was therefore necessary to develop a procedure for modifying the model to take into account a full range of impervious conditions. This involves several assumptions:

(a) For the Herring Run area, which is 11.9 percent impervious, equations can be derived representing unit flow from both the completely pervious and completely impervious portions of the basin.

(b) The yield from the completely impervious portions will be less than 100 percent of the rain that falls on them, since some water will flow from impervious surfaces to pervious surfaces where it will be lost to infiltration.

(c) While this percentage will actually vary, an average value can be calculated and used in the equations.

These assumptions were used in the derivation of the following equation, which is fully described in Appendix A:

$$Q_i = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right)\left[(B-KXI)R_i + CR_i^2 + DR_i^3\right] \qquad \text{(II-7)}$$

where:

Q_i = runoff/unit area from any Wilde Lake watershed on the i^{th} day

K = conversion factor between inches/day and cfs/mi^2 = 26.88

X = percent of rainfall actually running off impervious areas

I = fraction imperviousness of Herring Run = 0.119

I' = fraction imperviousness of any Wilde Lake watershed

R_i = rainfall on the i^{th} day

A,B,C,D = regression coefficients derived previously

The only unknown in this equation is X, the percentage of runoff from impervious areas. To obtain a value, the results of a study of effects of urbanization on runoff from selected Texas watersheds were used (Ref. 3). By relating increased runoff to increased imperviousness on the same watershed, the following equation was derived:

$$X = \frac{Q_I}{R}\left[\frac{1}{I} + \frac{I-1}{I(1+100I)^{0.339}}\right] \qquad \text{(II-8)}$$

where:

X = percent of rainfall actually running off impervious areas

Q_I = runoff volume (as calculated by the equation given in Appendix B) for an area with an impervious fraction of I

R = rainfall volume

I = fraction imperviousness

By taking individual storm data and calculating Q_I, a value of X can be calculated for each storm, and these values can be averaged over all storms. The value of $\overline{X}$ thus obtained is:

$\overline{X}$ = 0.8586

Using this value in the equation for Q_i, the final equation for runoff on the Wilde Lake watersheds becomes:

$$Q_i = A + R_i \left[23.08I' + \left(\frac{1-I'}{0.88}\right)(B - 2.77) \right] + R_i^2 \left[C\left(\frac{1-I'}{0.88}\right) \right] + R_i^3 \left[D\left(\frac{1-I'}{0.88}\right) \right] \quad \text{(II-9)}$$

Separate equations for each sub-watershed and for each season can be derived by using the appropriate values of I' and the regression coefficients A, B, C, and D, and from these equations the continuous runoff hydrograph can be calculated using a daily rainfall input.

To illustrate the actual effects of imperviousness on the runoff yield per unit area, an equation can be developed giving the percent increase in unit yield of an area with a fraction imperviousness of I' over the natural area (0 impervious fraction).

$$P = \frac{I' \left[KX - B - CR_i - DR_i^2 \right]}{\frac{A(1-I)}{R_i} - KXI + B + CR_i + DR_i^2} \quad \text{(Appendix C)} \qquad \text{(II-10)}$$

where P = percent increase in yield over the natural state. Figure II-8 and II-9 show respectively, P versus I' for different values of R_i and P versus R_i for different values of I'. The results of the Texas study on effects of urbanization are also indicated in Figure II-8. It can be seen that for every value of I', the precipitation values between about 0.2 inch and 1.0 inch have the greatest effects on P. This would be expected, since during small rainstorms the pervious areas would contribute very little runoff, and during heavy rains, the pervious areas would eventually contribute large volumes because the infiltration capacity would be exceeded.

5. Sensitivity Analysis

As one method of testing the usefulness of the final modified regression equation, a sensitivity analysis was made of the equation coefficients. Such an analysis indicates how sensitive the calculated values of runoff are to minor variations in these coefficients and also provides a measure of the reliability of the equation over the range of rainfall values. If it was determined, for example, that a 10 percent increase in the value of X in the regression equation would cause a 20 percent change in calculated runoff for rainfalls less than one inch, and an increasingly greater change for larger rainfall, then it could be stated that the regression equation is highly sensitive to the value of X and becomes unreliable at large rainfalls. This would then necessitate a much more careful determination of the value of X to be used in the equation.

To perform the type of sensitivity analysis required, the regression equation was partially differentiated in the terms of each coefficient, and the resulting differential equations were used to calculate the change in runoff as a function of the change in coefficients. This was done for both the upper and lower limits of the imperviousness parameter.

The partial derivatives of the equation are:

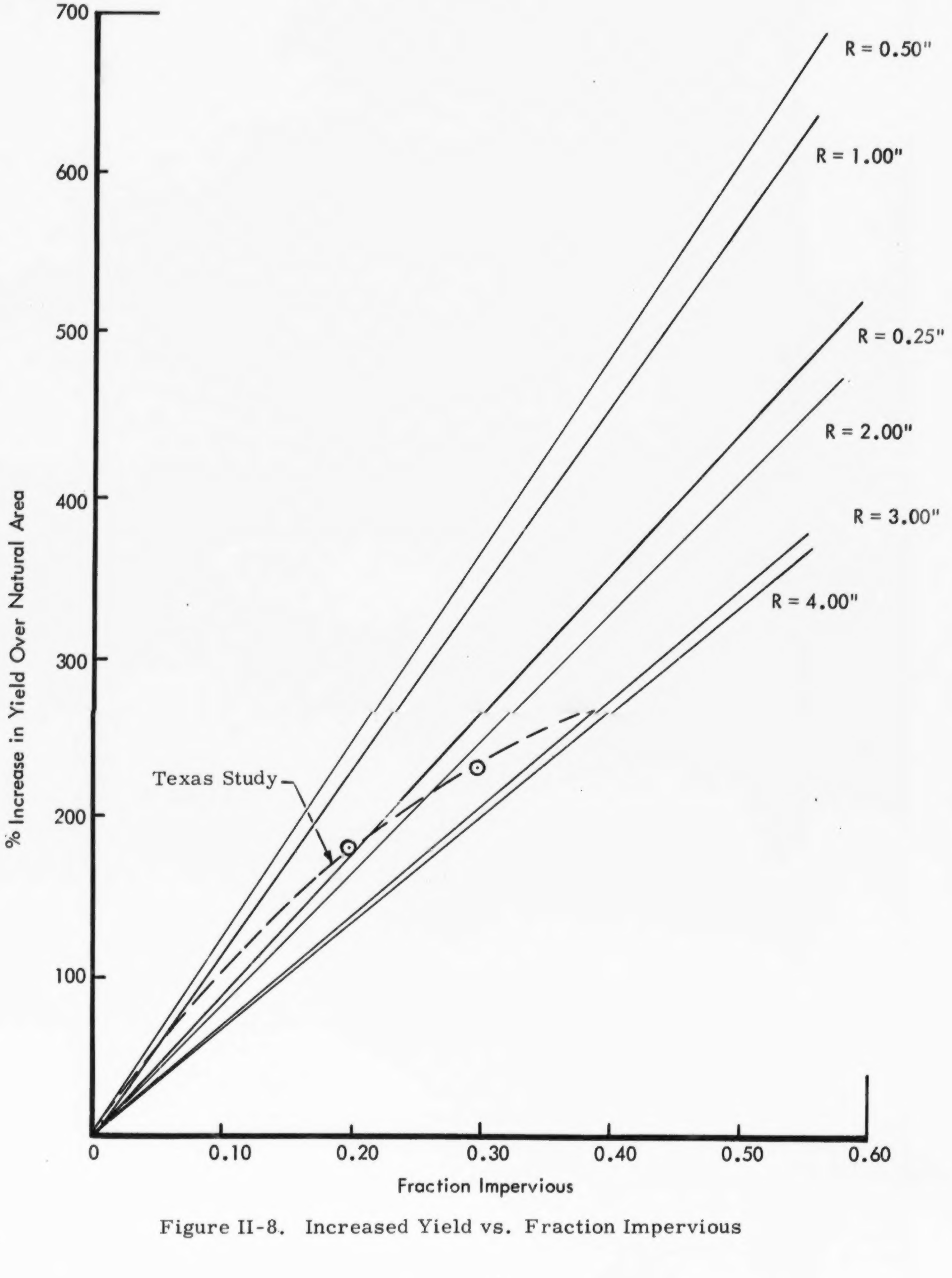


Figure II-8. Increased Yield vs. Fraction Impervious

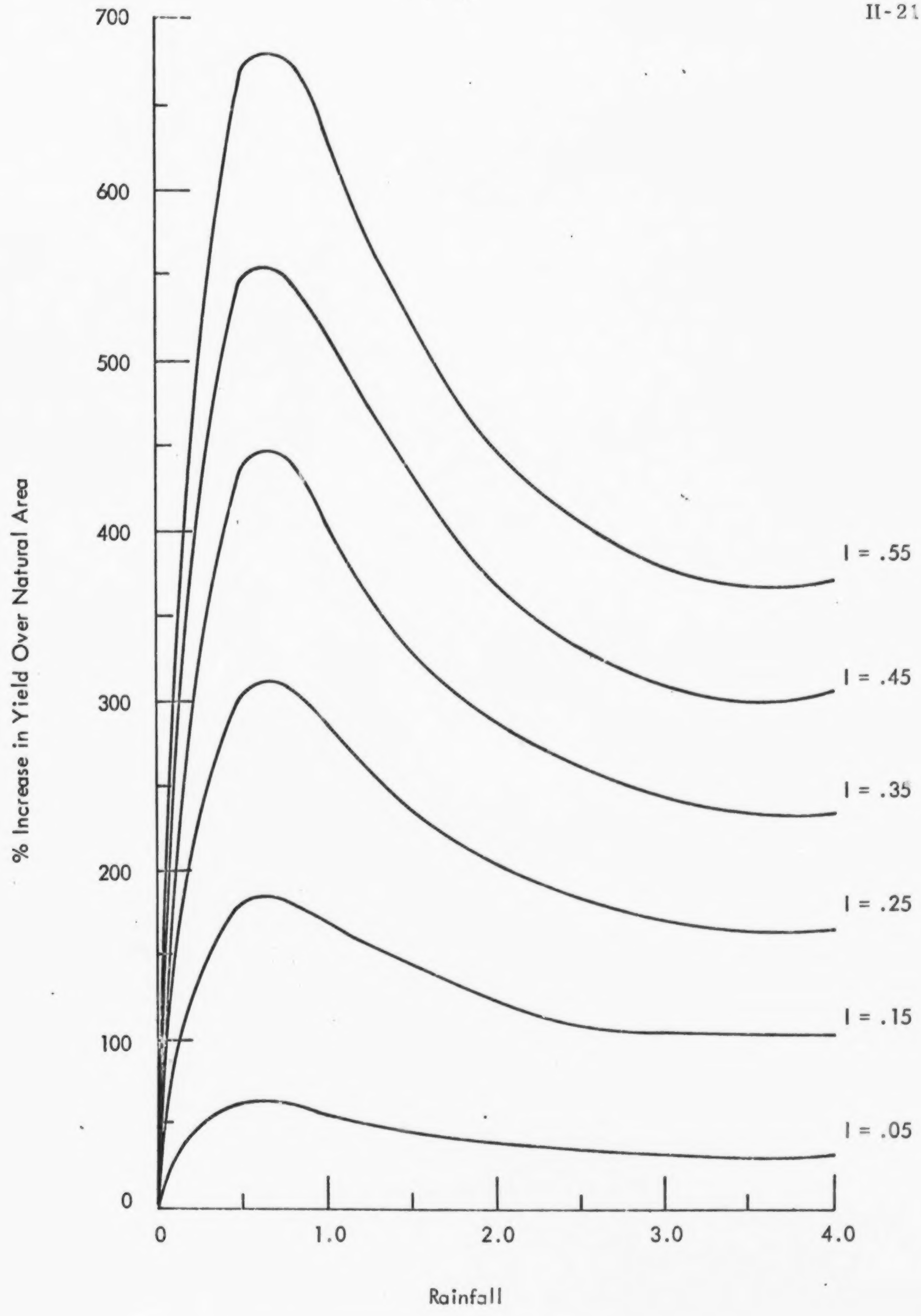


Figure II-9. Increased Yield vs. Rainfall Volume

$$\frac{\partial Q}{\partial A} = 1;\ \Delta Q = \Delta A \tag{II-11}$$

$$\frac{\partial Q}{\partial B} = \left(\frac{1-I'}{1-I}\right) R;\ \Delta Q = (\Delta B)R\left(\frac{1-I'}{1-I}\right) \tag{II-12}$$

$$\frac{\partial Q}{\partial C} = \left(\frac{1-I'}{1-I}\right) R^2;\ \Delta Q = (\Delta C)R^2\left(\frac{1-I'}{1-I}\right) \tag{II-13}$$

$$\frac{\partial Q}{\partial D} = \left(\frac{1-I'}{1-I}\right) R^3;\ \Delta Q = (\Delta D)R^3\left(\frac{1-I}{1-I}\right) \tag{II-14}$$

$$\frac{\partial Q}{\partial X} = \left(\frac{I'-I}{1-I}\right) KR;\ \Delta Q = (\Delta X)KR\left(\frac{I'-I}{1-I}\right) \tag{II-15}$$

Using the values of $I' = 0.565$ and $I' = 0.04$ and allowing R to range from 0 to 3, the value of ΔQ was determined for a 10 percent change in each coefficient. Plots were then made of the percent change in Q versus rainfall for each differential equation. These results are shown in Figure II-10 and II-11. It can be seen that the maximum change in Q for a 10 percent change in coefficients is only 12 percent, and in most cases the coefficient variation is much larger than the variation in runoff.

It was therefore determined that the final regression equation is not abnormally affected by coefficient changes, and throughout the rainfall range the equation remains stable.

6. Development of Storm Hydrograph and Peak Flow Model

In order to calculate the expected peak flows for a storm of any recurrence interval, a method was desired which could use the rainfall intensity-duration-frequency curves for Howard County as a basic input. Although the standard procedure for this type of calculation is to use the Rational Method with an appropriate runoff coefficient for each watershed, a more sophisticated approach which would provide more detailed information was selected. This approach, mentioned previously, involves the routing of a rainfall hyetograph through a linear storage reservoir model to obtain a complete storm hydrograph. It has been used with great success by the U.S. Department of Agriculture in analyzing storm runoff from small agricultural watershed (Ref. 2), and it required only minor changes to be adapted to this study.

As the first step in this analysis, the Howard County rainfall curves were fitted to an equation relating the intensity of rainfall, the duration of rainfall, and the return period of the storm. This equation, in a standard form is:

$$i = \frac{200T^{0.176}}{(t+25)^{1.1}} \tag{II-16}$$

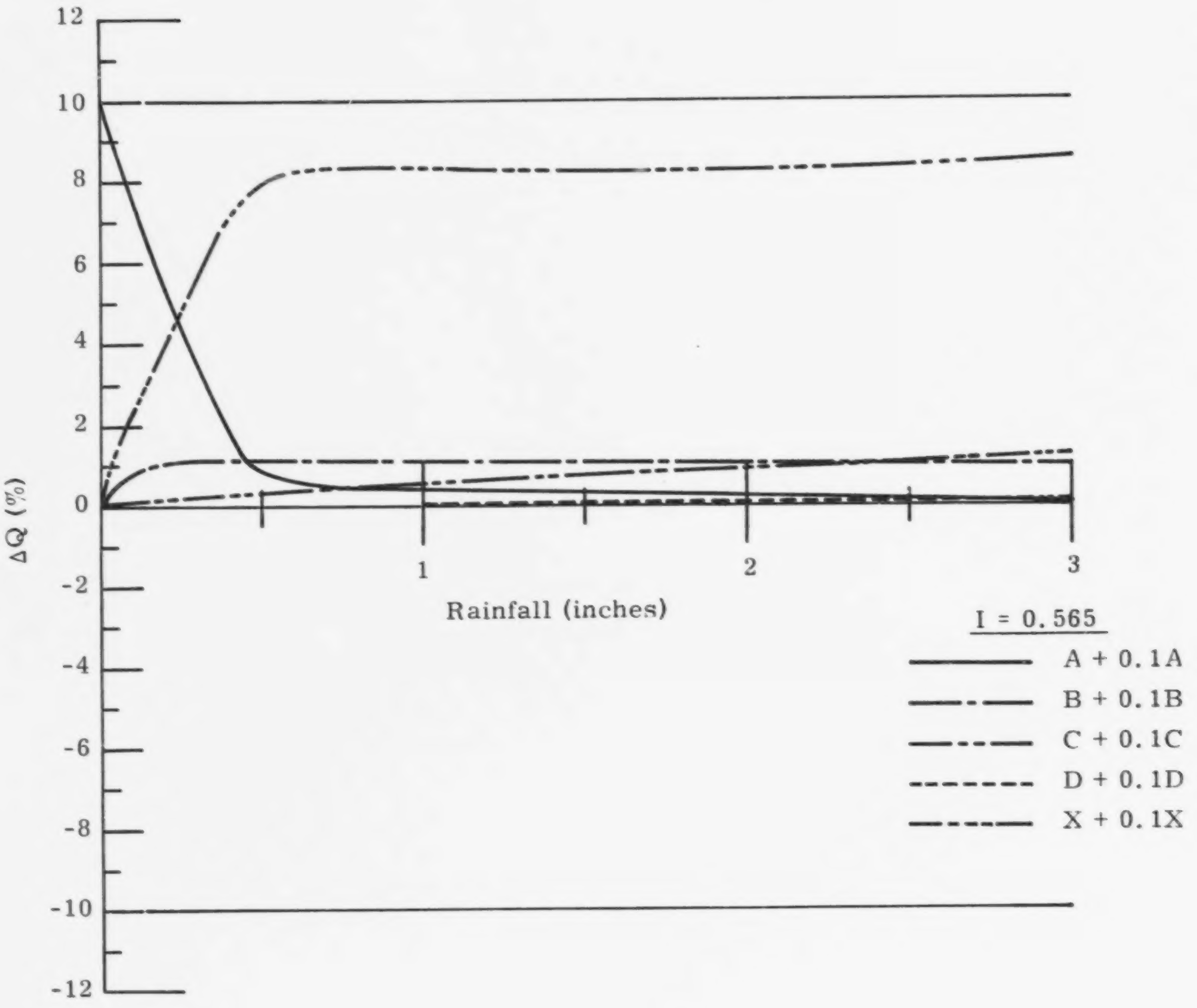


Figure II-10. Sensitivity Analysis - Fraction Impervious I = 0.565

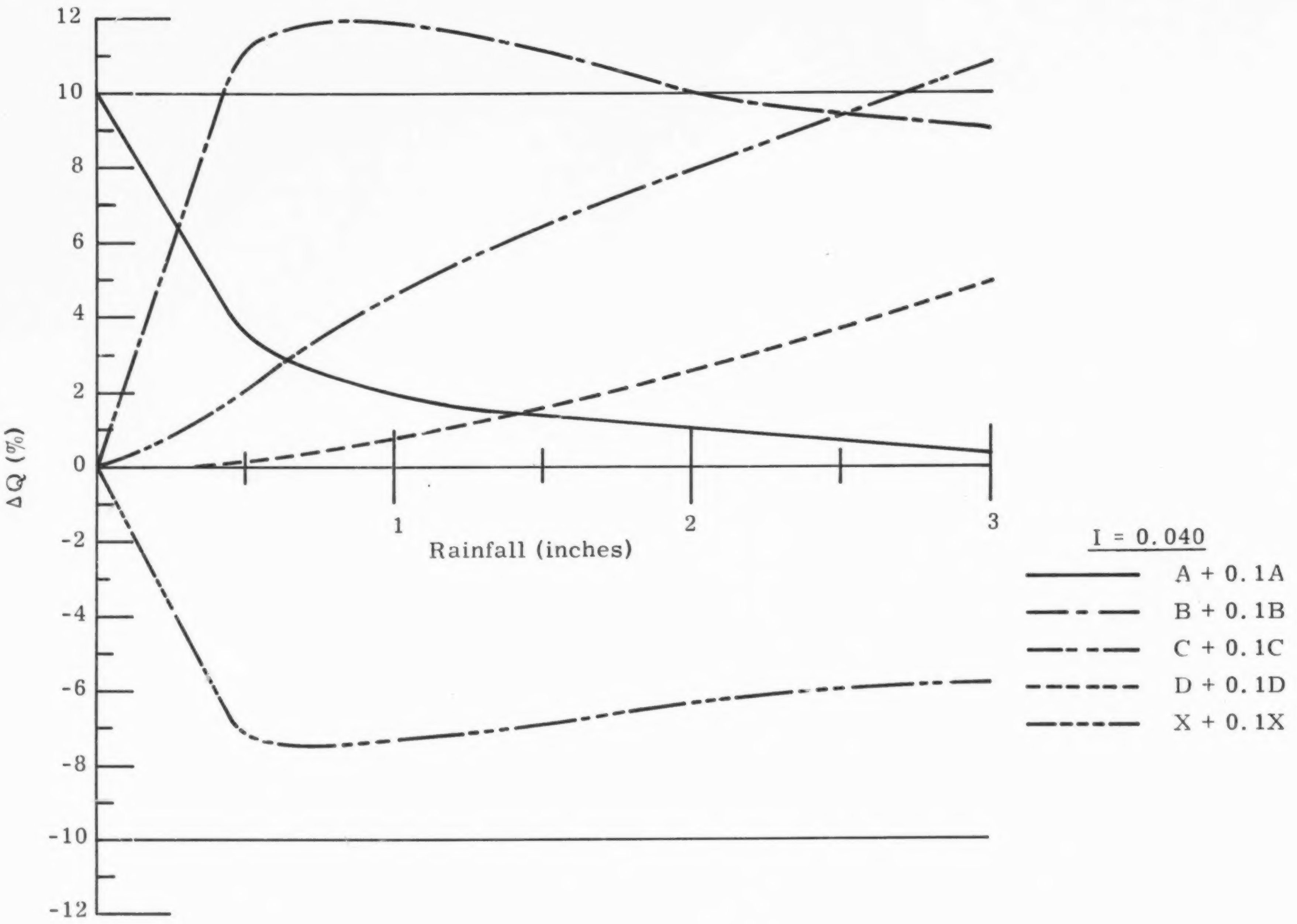


Figure II-11. Sensitivity Analysis - Fraction Impervious I = 0.040

where:

i = intensity (inches/hour)

T = return period (years)

t = duration of rainfall (minutes)

Using this equation, rainfall hyetographs for a 60 minute storm with return periods on one, two, five, and 10 years were synthesized. These hyetographs were developed with the following conditions:

(a) For each storm, the intensity of rainfall for any duration from five minutes to 60 minutes is identical to the intensity calculated by the rainfall equation.

(b) The hyetograph is approximately symmetric about the peak intensity.

(c) The minimum time interval for calculating intensities is five minutes.

In addition to these conditions, the 60 minute storm volume is only slightly less than the volume of a 24 hour storm of the same return period, and therefore, is analogous to the daily rainfall and runoff volumes used in the regression analysis.

The linear storage reservoir model, which calculates the runoff hydrograph, requires effective precipitation as an input. Since effective precipitation is simply the rainfall hyetograph minus any losses due to infiltration or surface storage, a method was needed to alter the hyetograph to include these losses. This was done by using an infiltration equation for the losses on the pervious portion of a watershed, and by assuming the rainfall on the impervious portion is entirely available for runoff. The chosen infiltration equation was one originally derived by Horton (Ref. 4), which had been used in a Johns Hopkins University study of the rainfall in runoff process (Ref. 5), and for which values of infiltration coefficients had been calculated for the Baltimore area. The steps involved in changing the original hyetograph are summarized in the following equations, which were applied to the hyetograph for each interval of Δt:

$$P_{Per} = P_{Imp} - f = P_{Imp} - \left[f_o + (f_o - f_c)(1 - e^{-at}) \right] \qquad \text{(II-17)}$$

where:

f = infiltration loss = $f_o + (f_o - f_c)(1 - e^{-at})$

f_o = initial infiltration rate (inches/hour) = 2.5

f_c = final infiltration rate (inches/hour) = 1.5

a = exponential decay constant (hour^{-1}) = 0.7

P_{Per} = effective rainfall on pervious areas

P_{Imp} = rainfall on impervious areas = original hyetograph

$$P_{Per} = 0 \text{ when } f > P_{Imp} \tag{II-18}$$

$$P_{Eff} = P_{Imp}\,(I) + P_{Per}\,(1-I) \tag{II-19}$$

where:

P_{Eff} = final effective precipitation hyetograph

I = fraction imperviousness of the watershed

The resulting effective precipitation hyetograph has applied as input to a linear storage reservoir model of the runoff process. This model uses the following continuity equation as its basis:

$$I = \frac{q_1 + q_2}{2} + \frac{S_2 - S_1}{\Delta t} \tag{II-20}$$

where:

I = average inflow rate for time period Δt(cfs)

q = outflow rate (cfs)

S = storage volume (feet^3)

Δt = time increment between times 1 and 2

The effect of storage on the outflow depends on the physical properties of the entire drainage area, and on the volume of storage. Normally the storage will impart a time delay to the outflow with respect to the inflow into the basin. This concept is expressed by the equation:

$$I - q = M \frac{\Delta q}{\Delta t} \tag{II-21}$$

where M is the basin storage coefficient, which may be considered to be equal to the lag time (2).

In order to obtain the best estimates of peak flow, timing, and hydrograph shape, investigations have shown that two routings of the rainfall through half of the indicated storage are required, where the output from the first reservoir becomes the input to the second.

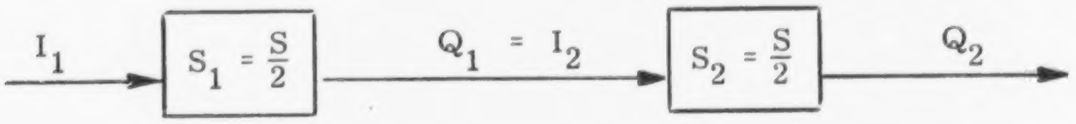


Figure II-12. Routing Method - Linear Storage Reservoir Model

From the two preceding equations, and the double-routing technique, a single equation can be derived to take effective precipitation as the input to the first reservoir and calculate the output from the second reservoir.

This final equation for routing the effective precipitation is:

$$Q_i = \left(\frac{\Delta t}{m+\Delta t}\right)^2 \left(P_{Eff_i} + 2P_{Eff_{i-1}} + P_{Eff_{i-2}}\right) + 2\left(\frac{m-\Delta t}{m+\Delta t}\right) Q_{i-1} - \left(\frac{m-\Delta t}{m+\Delta t}\right)^2 Q_{i-2} \qquad \text{(II-22)}$$

where:

Q_i = runoff in cfs/acre during the i^{th} time interval

m = lag time (minutes)

Δt = time interval (minutes)

Using a computer, the effective precipitation hyetographs for each storm return period, and for a range of imperviousness from 0.1 to 0.5, were used as input to the routing equation. Various lag times from four minutes to 20 minutes were assumed and a total of 48 hydrographs representing each combination of return period, imperviousness, and lag time was obtained as output.

These data were then reduced by calculating for each hydrograph the maximum average flow for durations from five minutes to 60 minutes, or in equation form:

$$Q_d = \text{Max}\left(\sum_d Q/d\right), \text{ where } d = \text{duration in minutes} \qquad \text{(II-23)}$$

The resulting values of Q_d, plotted against duration (for every combination of lag time, imperviousness, and return period), provided a set of curves that are analogous to the rainfall intensity-duration-frequency curves. The evolution of these runoff curves is summarized in graphical form in Figure II-13, and the effects of all the parameters on Q can be seen in Figure II-14 through II-16.

In order to more easily use the data that these curves represent, an equation was graphically fitted to all of the points for durations up to 30 minutes. This equation related all of the parameters, eliminating the need for interpolation between the original curves, and allowing a rapid determination of peak flows for any sub-watershed. The equation which follows has a multiple correlation coefficient of 0.98 when compared to the original data points.

$$Q_d = 8T^{0.301}(m+3)^{-0.732} - 16T^{0.246}(m+10)^{-2.200}d + 0.955T^{0.033}I(2.482-0.129m+0.010m^2-0.00033m^3) \qquad \text{(II-24)}$$

where:

m = lag time (minutes) $4 \leq m \leq 20$

d = duration (minutes) $5 \leq d \leq 30$

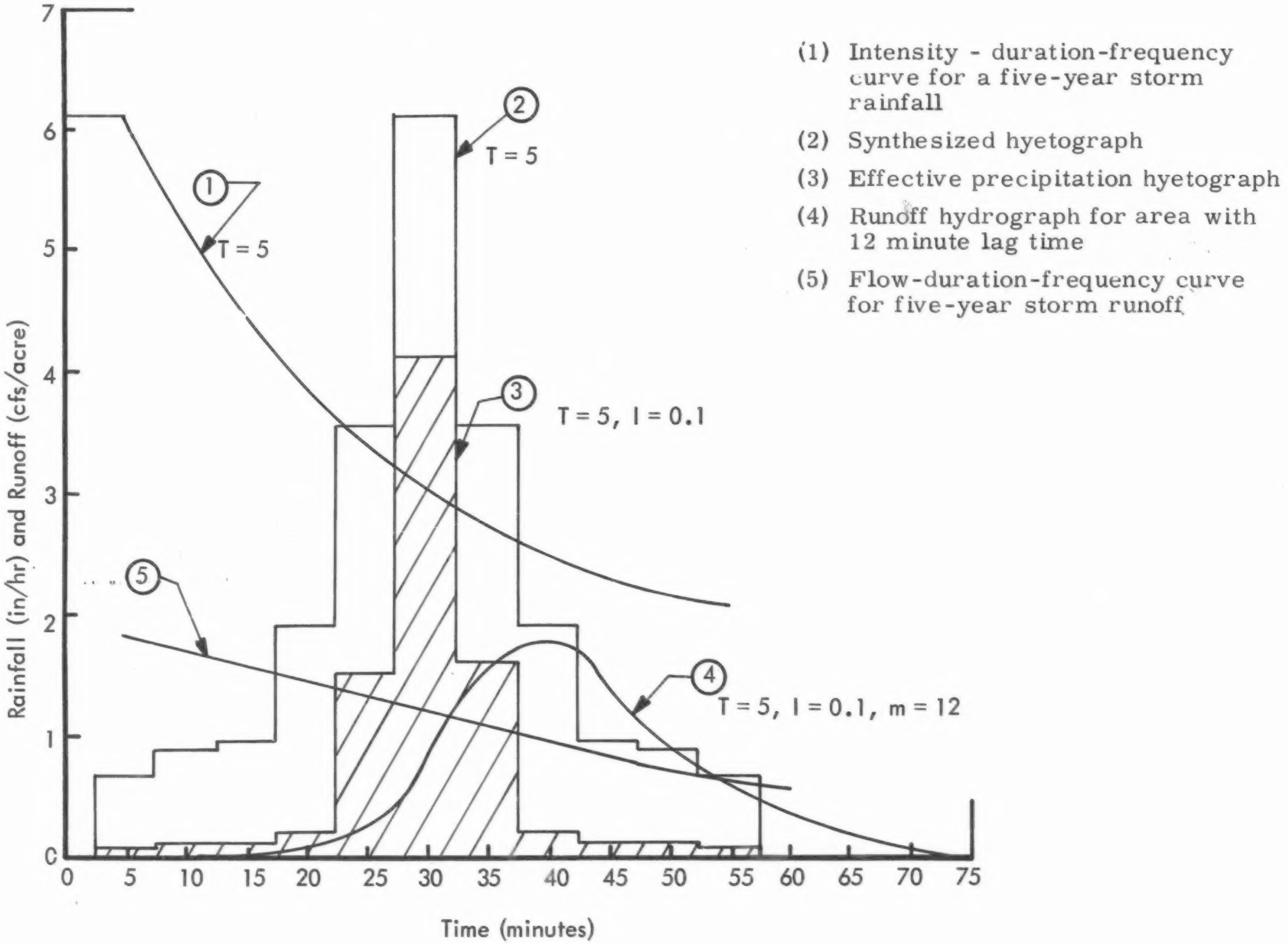


Figure II-13. Derivation of Flow-Duration-Frequency Curve for 5-Year Storm Runoff

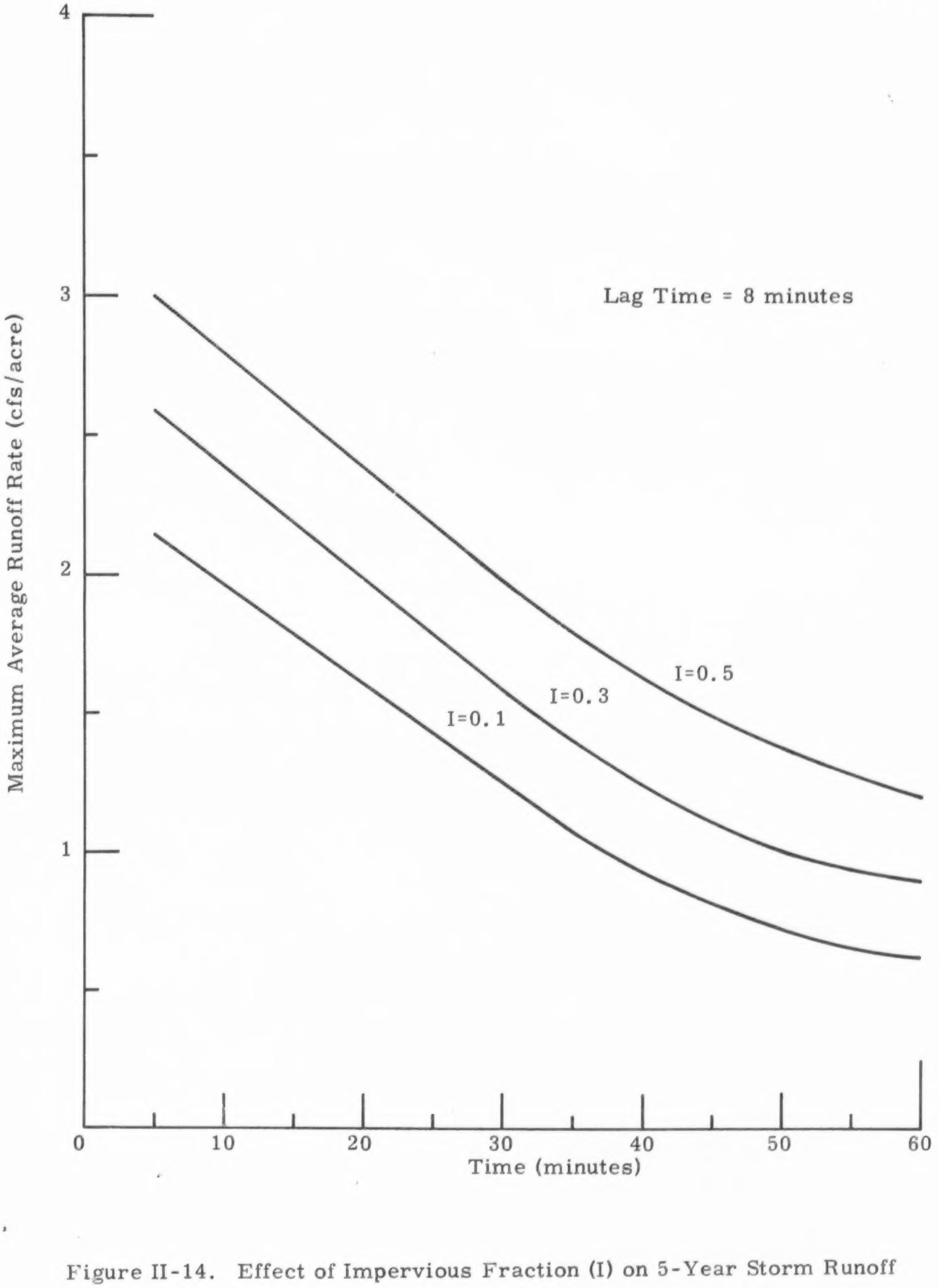


Figure II-14. Effect of Impervious Fraction (I) on 5-Year Storm Runoff

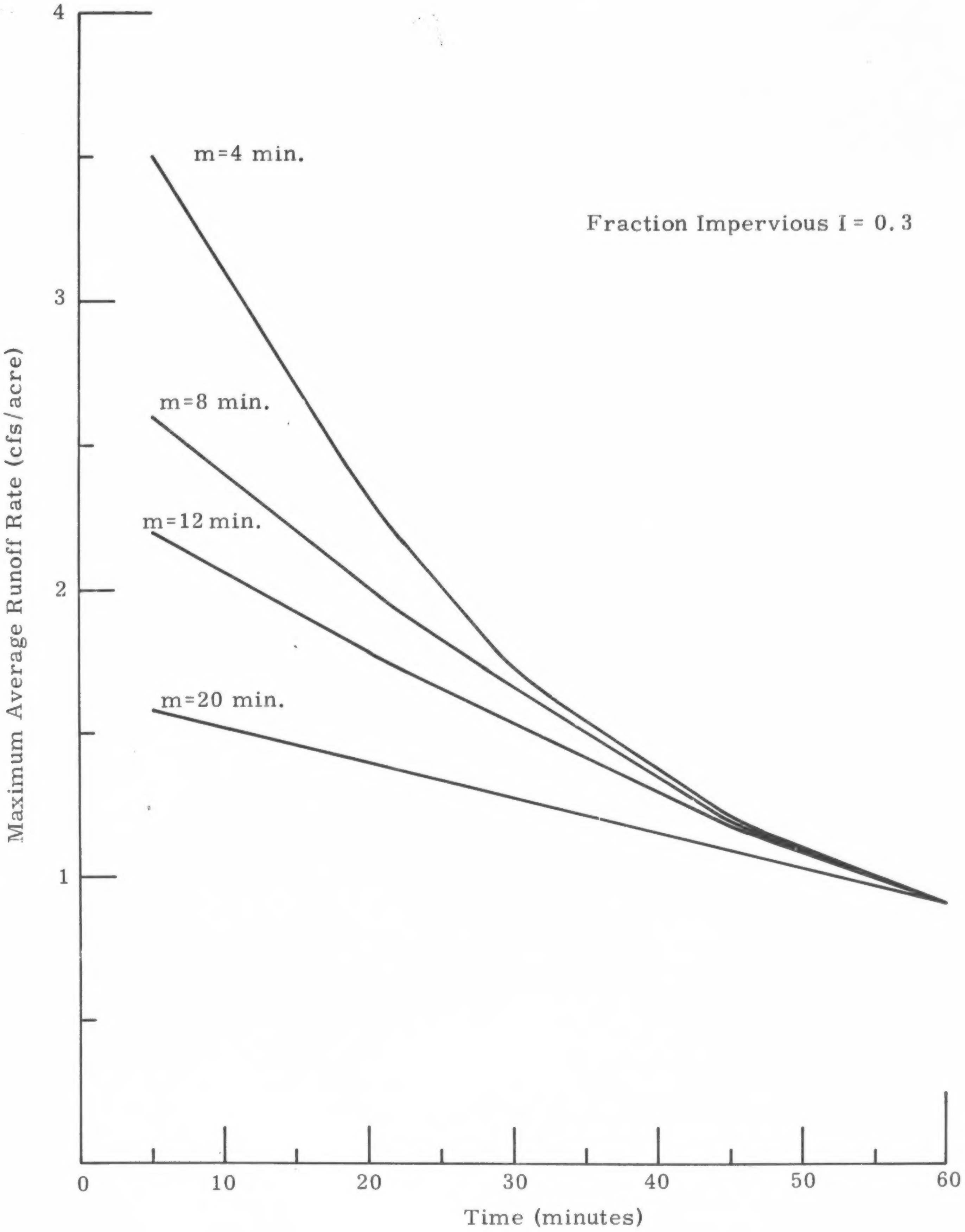


Figure II-15. Effect of Lag Time (m) on 5-Year Storm Runoff

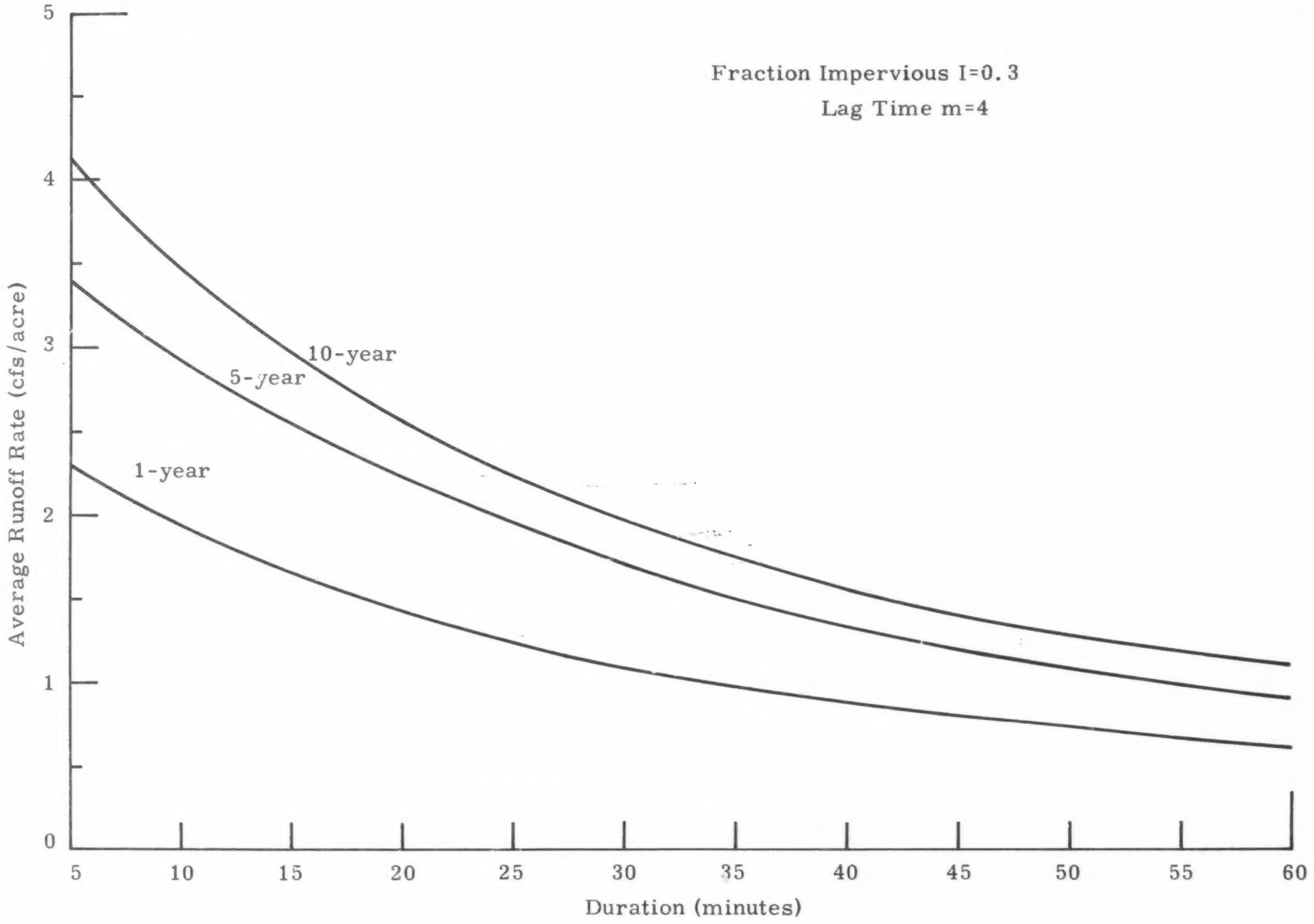


Figure II-16. Average Runoff Rate vs. Storm Duration

T = return period (years) $1 \leq T \leq 10$

I = impervious fraction $0.1 \leq I \leq 0.5$

By applying this equation to the sub-watersheds, storm flow determinations could be made for any storm up to 10 years in magnitude, and the results of these determinations and their application to sedimentation requirements are described in a subsequent section.

As a final application of this storm hydrograph analysis, the volumes of the individual hydrographs were calculated and plotted against imperviousness and return period. These runoff volumes represent approximately the maximum runoff volumes of a 24 hour storm for each return period, since the original 60 minute rainfall hyetograph volume was almost identical to the 24 hour rainfall volume. A graph of storm volumes is shown in Figure II-17.

7. Comparison Between the Regression Model and the Storm Hydrograph Model

Since two entirely different techniques were used to analyze storm water runoff, a comparison was made of the results of the two. Because both methods ultimately calculated the volumes of runoff, storm volumes were chosen as the basis for comparison.

A five year computer calculation of daily flows for the Wilde Lake watershed, as determined by the regression model, was examined and a frequency distribution of the runoff was plotted. This was then converted to a plot of daily flow versus return period as shown in Figure II-18. To calculate the volumes of storms using the hydrograph approach, curves similar to those in Figure II-17 were employed, using a fraction imperviousness of 0.22. This provided storm volumes for storms with return periods of one, two, five, and 10 years, which could be compared to the corresponding volumes in Figure II-18.

A plot of storm volumes calculated by both methods is given in Figure II-19. It can be seen that the different techniques gave approximately the same results with a maximum difference of only eight percent for a two year storm.

8. Input to the System Analysis

a. Regression Model. In order to use the final regression equation (Equation II-9) to calculate continuous runoff hydrographs for each of the Wilde Lake sub-watersheds, it was necessary to first determine the fraction imperviousness of each watershed. Since land use data was readily available for the Columbia project, a tabulation was made of the acreage that was to be devoted to each type of land use in every sub-watershed. A factor was applied to each use to represent the percent imperviousness estimated for that type of use. By multiplying the impervious factor b, the number of acres, and summing for all land uses, the total number of pervious and impervious acres could be obtained.

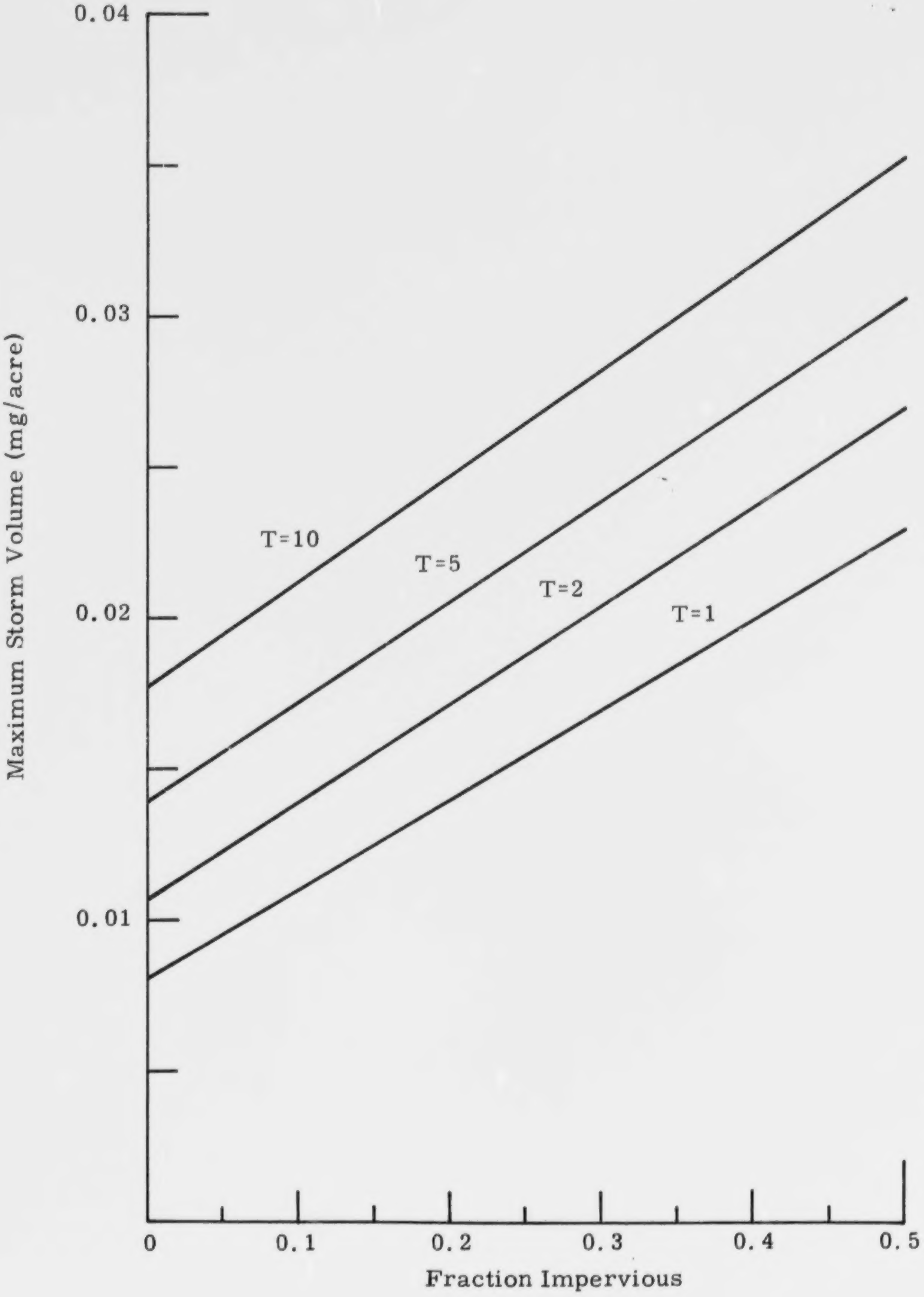


Figure II-17. Maximum Storm Volumes

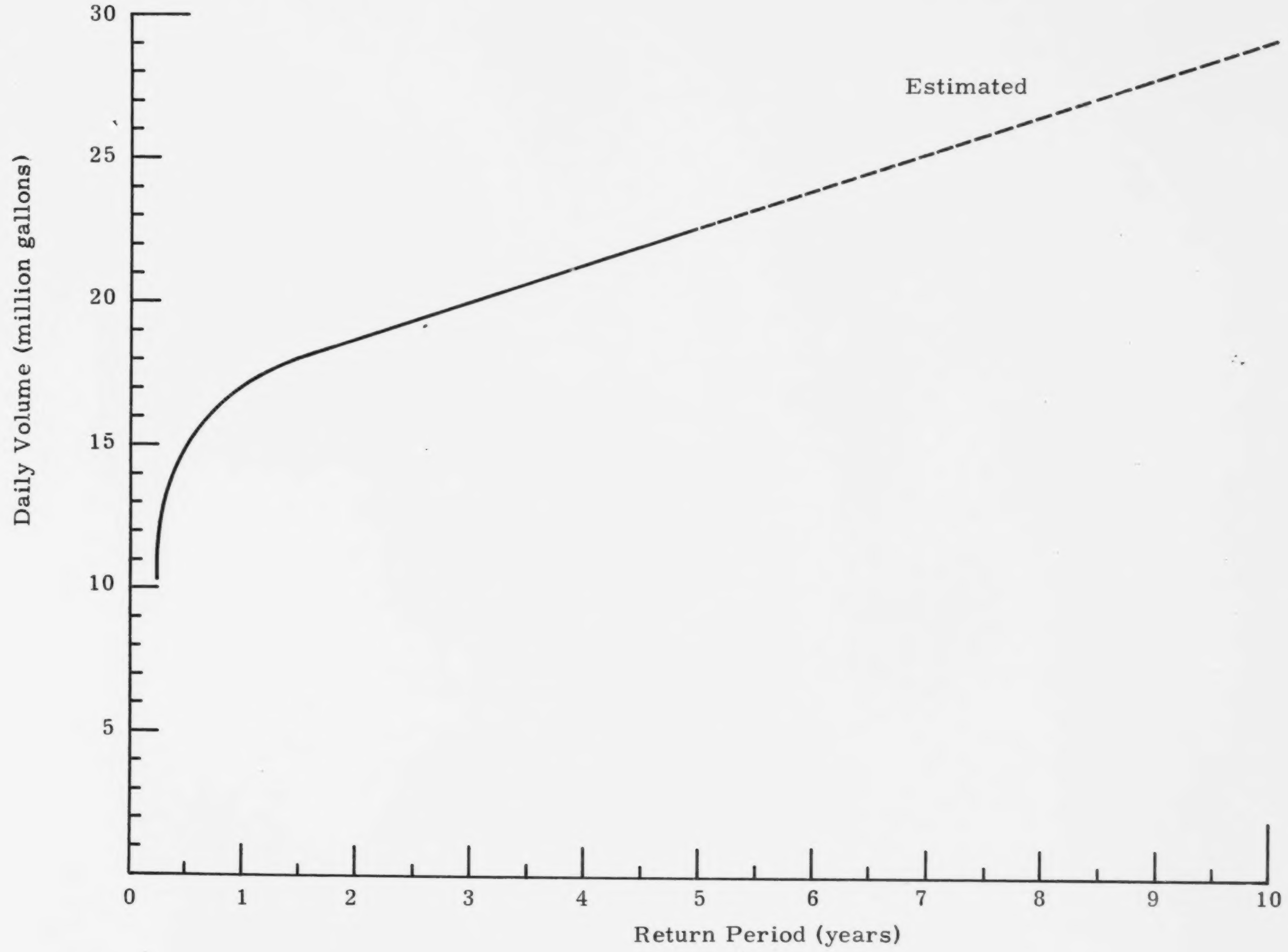


Figure II-18. Actual Daily Volume vs. Return Period - Wilde Lake

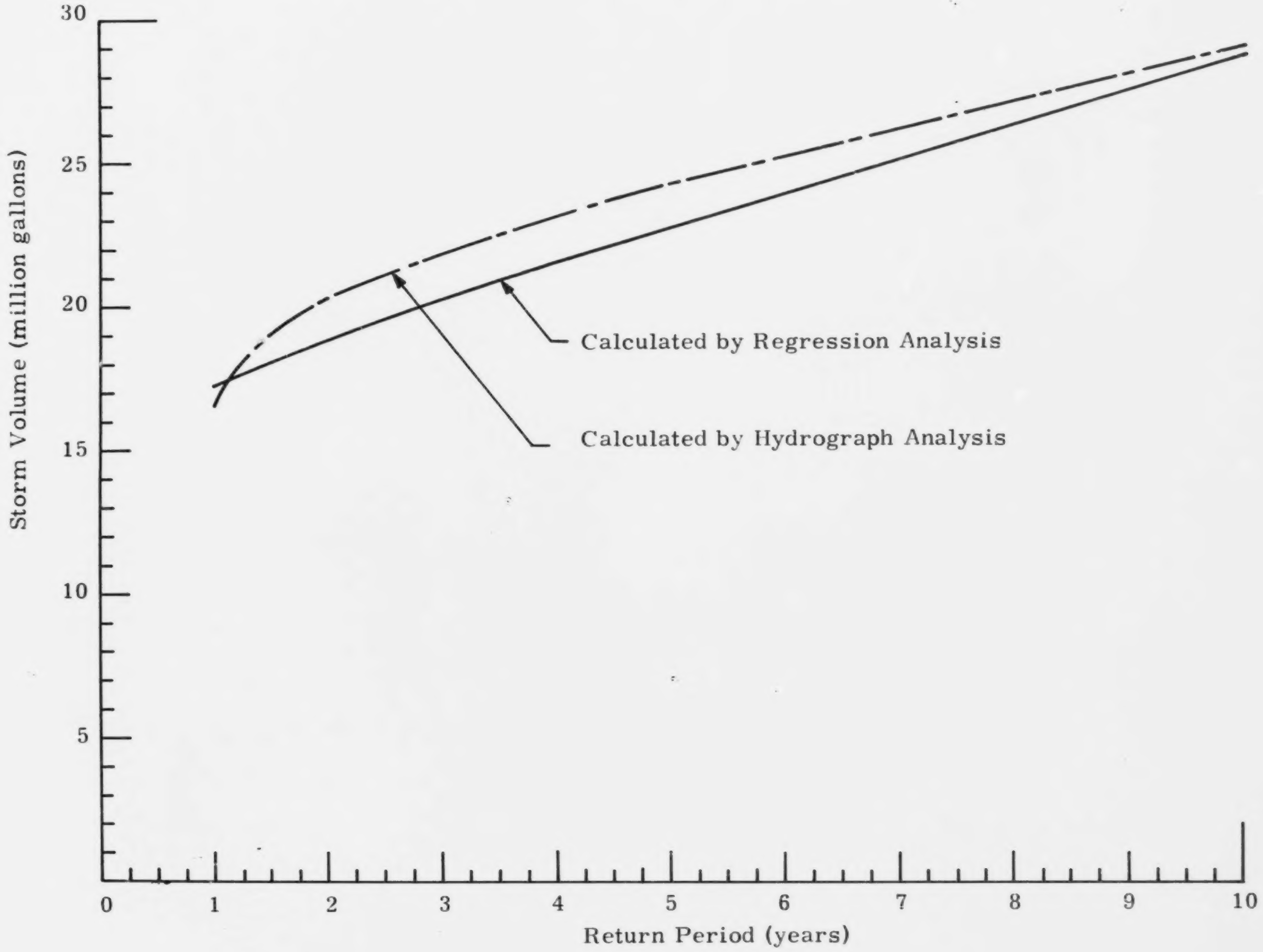


Figure II-19. Calculated Daily Volume vs. Return Period - Wilde Lake Watershed

For the portions of watersheds that were situated outside of the Columbia boundaries, a similar procedure was followed. In this case, either existing land uses or zoning maps were used to tabulate the land uses. The same impervious factors were applied to the non-Columbia acreage as were used for similar uses in the Columbia project.

Figure II-20 is an actual tabulation of land uses, residence units, population, and imperviousnesses for sub-watershed number 17. The same tabulation was prepared for all other watersheds, and the overall imperviousness (shown in the last line of Table II-3) was used as I in Equation II-9.

The only additional data required to calculate runoff hydrographs was daily precipitation for the Wilde Lake area. After examining monthly precipitation data from four rain gages in the vicinity of Columbia, it could be seen that the Friendship Airport weather station data agreed most closely with the monthly rainfall calculated for Columbia by the Thiessen method. Therefore, five years of daily rainfall data were obtained from Friendship Airport to be used as input to the regression model.

This data was first altered to more closely represent the long-term precipitation record at Friendship Airport. To perform the alteration, a simple normalization technique was used which preserved the historical time distribution of rainfall, but which changed the daily volume of rainfall on a random basis. This alteration eliminated the extremely dry or wet months, which on the average would be unlikely to occur in a given five year period, without eliminating most daily extremes or greatly changing yearly totals. The procedure used involved the following data input:

(1) A set of 30 monthly rainfall totals to represent typical monthly rainfalls expected. These were obtained from a 50 year distribution of monthly precipitation at the Baltimore Customs House. These values are shown in Table II-3.

(2) The continuous daily rainfall record for Friendship Airport expressed in inches for the five year period 1960 to 1964.

A computer program was used to take each month's daily rainfall data and sum these data to give the total monthly rainfall. This monthly value (M_i) was compared to the set of 30 typical monthly values, and the month that was closest to M_i numerically was chosen. (This can be called T_i.) The difference between M_i and T_i was calculated, and M_i was increased or decreased by this amount so that it equalled T_i. The adjustment was done in the following manner:

(1) If M_i was greater than T_i, one day during the month was chosen at random and 0.01 inch of rainfall was subtracted. (If that day had no rainfall, then another day was picked.) This was done repeatedly until M_i=T_i.

(2) If M_i was less than T_i, one day during the month was chosen at random and 0.01 inch of rainfall was added. (Once again, only days on which rainfall actually occurred were changed.) This was done repeatedly until M_i=T_i.

Project 3519

SUB-DRAINAGE AREA DATA SHEET

Coll. Area No. 17

	LAND USE	AREA (Ac.)	UNITS/ Ac.	RESID. UNITS	POPUL./ UNIT	POPUL.	% IMPERV.	IMPERV. AREA (Ac.)
COLUMBIA PROJECT	OPEN SPACE	10.7					0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Single Family - Medium Density	12.5	4.0	50	4.0	200	16.0	2.0
	Town-Houses	4.9	11.0	54	3.5	189	33.0	1.6
	Garden Apartments	6.8	17.0	116	2.5	290	20.0	1.4
	Mid-Rise Apartments	3.6	28.0	101	2.5	253	22.0	0.8
	Employment Centers	7.4					100.0	7.4
	School Sites	2.5					18.0	0.5
	Public Rights-of-way	9.5					85.0	8.1
NON-COLUMBIA	OPEN SPACE	0.1					0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Public Rights-of-way						8.5	
	COLUMBIA TOTAL	57.9		321		932	37.7	21.8
	NON-COLUMBIA TOTAL	0.1						
	AREA TOTAL	58.0		321		932	37.6	21.8

Figure II-20. Work Sheet - Typical Sub-Watershed Land Use Tabulation and Imperviousness Factor Calculation

TABLE II-3. TYPICAL MONTHLY RAINFALL TOTALS BALTIMORE CUSTOMS HOUSE, 1900-1950

Month	Total Monthly Precipitation (Inches)
1	0.40
2	0.69
3	0.92
4	1.18
5	1.40
6	1.52
7	1.71
8	1.85
9	2.00
10	2.15
11	2.29
12	2.41
13	2.59
14	2.71
15	2.90
16	3.05
17	3.21
18	3.41
19	3.60
20	3.80
21	4.01
22	4.28
23	4.50
24	4.80
25	5.10
26	5.50
27	6.00
28	6.65
29	7.45
30	10.70

As a numerical example of this, assume that for January of 1962 a total of 5.02 inches of rain fell according to the sum of the first 31 daily volumes. It is found by comparison to the 30 typical monthly values that the typical month which is numerically the closest to 5.02 inches is one with 5.50 inches of rain. This value therefore represents T_i.

$$T_i - M_i = 0.48 \text{ inches} \tag{II-25}$$

Since M_i is less than T_i, the 0.48 inches must be added to the daily rainfall during the month of January 1955. A day on which rainfall occurred would be picked at random and 0.01 inch would be added to the rainfall data for this day. This procedure would have to be done 48 times until the original 5.02 inches of January rainfall would equal 5.50 inches.

This procedure was followed for each of the 60 months in the five year record, and the adjusted daily precipitation was then used as the input to the regression model. The computer was used to output daily runoff values (neglecting base flow) for each sub-watershed and combination of watersheds. A typical year's output for one of the sub-watersheds, including coefficients, is shown in Figure II-21.

It should be noted that the five years of rainfall data, when compared to the long term record, consist of one very wet year, three dry years, and one average year. The occurrence of below average precipitation during three years should provide a conservative input to an economic study of storm water reuse.

b. Storm Volumes. To provide the estimates of one year storm volumes for different watersheds, the results of the hydrograph analysis were employed. For every sub-watershed, the values of imperviousness and size were used to obtain from Figure II-17 the total volumes of a one year storm. These values were then tabulated and are presented in Table II-4. The application of these volumes to the sizing of reservoir ponds and sedimentation facilities will be discussed in a subsequent section.

9. Gaging Station Results

A temporary stream gaging station was installed in one of the sub-watersheds on which little development had taken place. This gaging station had two primary objectives:

(a) To provide streamflow data to verify the regression model which was used to calculate runoff.

(b) To provide water quality data to compare with the data provided by the literature search.

The watershed that was selected for gaging is a 130 acre area in the upper end of the Wilde Lake drainage basin and is scheduled for development during the end of 1968. At the time of installation, several roads had been completed, giving the area an imperviousness of six percent. The undeveloped portion of the area consists of woods and a meadow with two main stream channels joining several hundred feet above the gage site (Figure II-22).

```
RUN-OFF DATA       RUN NO. = 6        AREA = 0.1078 SQ.MILES

REGRESSION COEFFICIENTS

                  A          B          C          D                AREA =     0.1078
      JAN   0.0        6.6134     0.6777     0.0827
      FEB   0.0        6.6134     0.6777     0.0827
      MAR   0.0        6.6134     0.6777     0.0827
      APR   0.0        5.4347     1.6078    -0.3519
      MAY   0.0        5.4347     1.6078    -0.3519
     JUNE   0.0        5.4347     1.6078    -0.3519
     JULY   0.0        5.2710     1.6515    -0.2183
      AUG   0.0        5.2710     1.6515    -0.2183
     SEPT   0.0        5.2710     1.6515    -0.2183
      OCT   0.0        4.8080     3.8510    -1.1654
      NOV   0.0        4.8080     3.8510    -1.1654
      DEC   0.0        4.8080     3.8510    -1.1654

          JAN.      FEB.      MAR.      APR.       MAY      JUNE      JULY      AUG.     SEPT.      OCT.      NOV.      DEC.
   1      0.69      0.05      0.09      0.01      0.25      0.09
   2                                              0.02                                              0.15
   3                0.95                                    0.07      0.26                0.03      0.06
   4                0.10      0.06                                                        0.98
   5
   6                          0.04                0.12                0.36      0.42
   7                                              0.32                0.02                                    0.27
   8                0.70      0.66                          0.10
   9                                    0.23      0.34      0.77      0.01      0.15                                    0.10
  10                                    0.67                0.02                                                      0.06
  11                                              0.44                          0.06                                  0.06
  12                0.17                0.46      0.04                          0.04                                  0.45
  13                0.02      0.12      1.03                0.13      0.21
  14                          0.09                          1.29      0.08                0.01      0.80      0.07
  15      0.53                                              0.02                          0.08
  16      0.05                          0.16                          0.90      0.04                          0.07      0.08
  17                                                                  0.07                                    0.04      0.42
  18                0.32      0.39                                                                                      0.44
  19      0.62                0.18                                                                            0.06
  20      0.05                                                                  0.23      0.05                0.09
  21                0.12                                    0.51                0.06                1.89
  22                0.46      0.54      0.09                                                        0.02
  23      0.03      0.29      0.12                                    0.05      1.15                          0.15      0.07
  24                                                                  1.39                                    0.57      0.13
  25                0.34                0.02                0.46                0.49
  26      0.27      0.02                0.14      0.22      0.02                0.36      0.02
  27                                                        0.22
  28                0.09      0.04      0.21                                                                            0.07
  29                                    0.03      0.04                0.01
  30                                                        0.02                                    0.01
  31                          0.64
```

Figure II-21. One Year Daily Hydrograph for Sub-Watershed #6

TABLE II-4. CALCULATED ONE YEAR STORM VOLUMES FOR WILDE LAKE SUB-WATERSHEDS

Watershed	Storm Volume (Million Gallons)
1	0.461
2	0.586
3	0.530
4	0.342
5	0.530
6	1.104
7	0.803
8	0.626
9	1.460
10	1.258
11	0.854
12	0.594
13	0.697
14	1.030
15	0.236
16	0.782
17	1.130
18	0.290
19	0.924
20	1.203
21	0.690
22	0.358

The instrumentation (Figure II-23) consisted of a compound weir located at the end of a long reach of a straight channel approximately six feet wide and two and one-half feet deep. When the weir plate was put into place, the water formed a distinct pond for a distance of 40 feet upstream.

Stage measurements of discharge over the weir were made using a stilling well, float, and Stevens A35 water level recorder. This recorder was located in an instrument box and was powered by a storage battery and power inverter. It provided a continuous record of water level with both good time and stage resolution.

The stage measurements were related to discharge over the weir by the use of an equation derived by model studies conducted by the Bureau of Reclamation. Field measurements were made of the flow by timing the rate of fill of known-volume containers, and these measurements closely agreed with the values calculated by the equation.

Approximately 40 feet upstream from the end of the weir pond an automatic water sampler was installed. This took water samples at preset time intervals and was usually hand started whenever rain was forecasted. These samples were collected and returned to the laboratory for analysis.

A University of Maryland weather station located a short distance from the watershed provided total rainfall information to be used in the regression model.

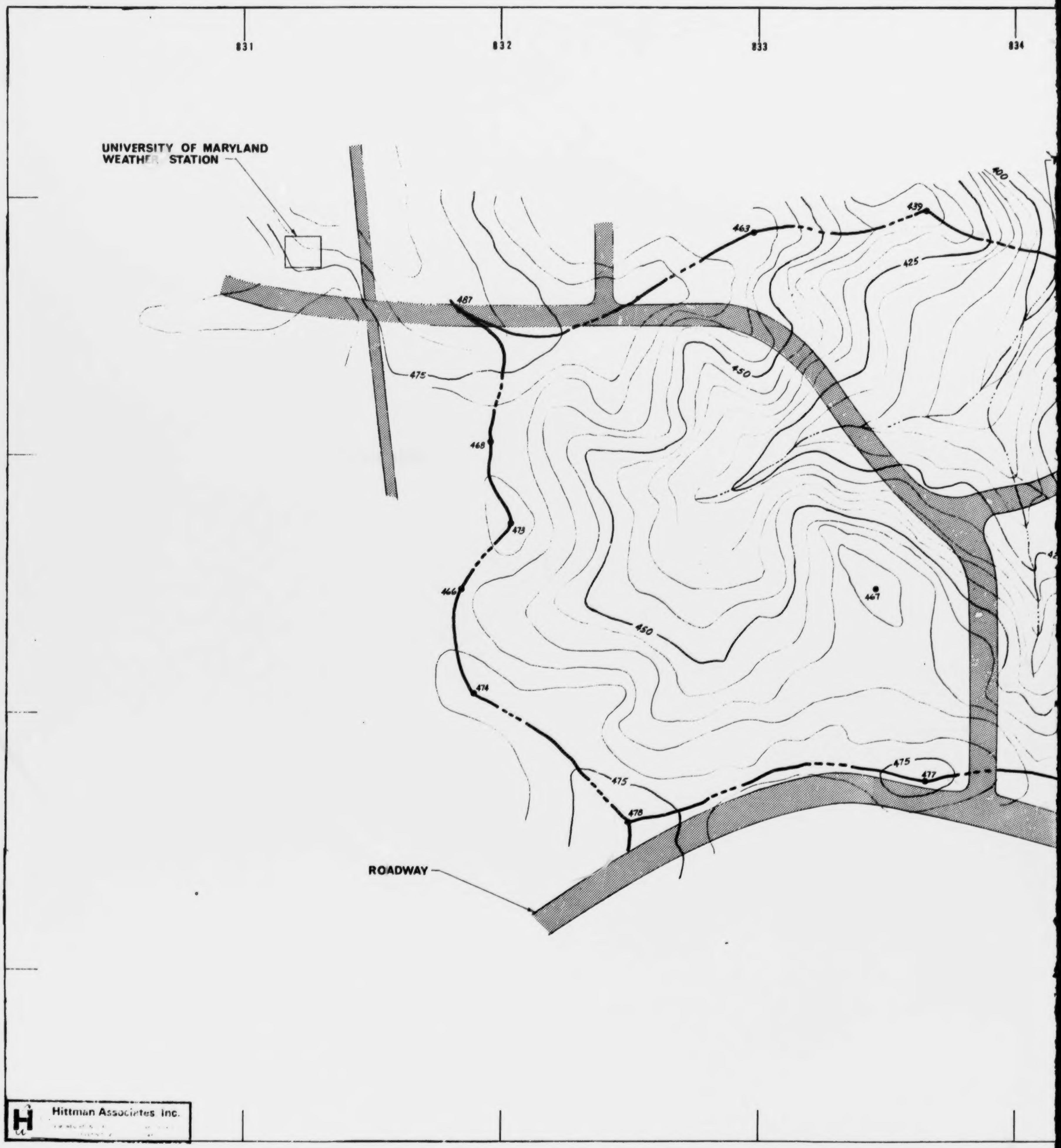
During the time of operation of the gaging station, three storms occurred from which runoff hydrographs were obtained. These three hydrographs are shown in Figures II-24 through II-26. From the hydrographs, the volume of runoff was determined and compared to the runoff volume calculated by regression model for the same amount of rainfall. Table II-5 gives the results of this comparison.

TABLE II-5. COMPARISON OF GAGED AND CALCULATED RUNOFF

Storm	Date	Rainfall (inches)	Measured Runoff (cfs-days)	Calculated Runoff (cfs-days)
1	6/26-6/27	0.79	0.242	0.352
2	6/27-6/28	0.66	0.236	0.254
3	7/19	1.16	0.70	0.60

Correlation between measured and calculated = 0.92

For the first and last storms, water samples were also collected. The results of the analysis of these samples, as well as the analysis of base flow, will be presented in a subsequent section.


831
832
833
834
UNIVERSITY OF MARYLAND WEATHER STATION
400
439
463
425
487
450
475
468
473
466
467
450
474
475
478
475
477
ROADWAY
Hittman Associates Inc.

A

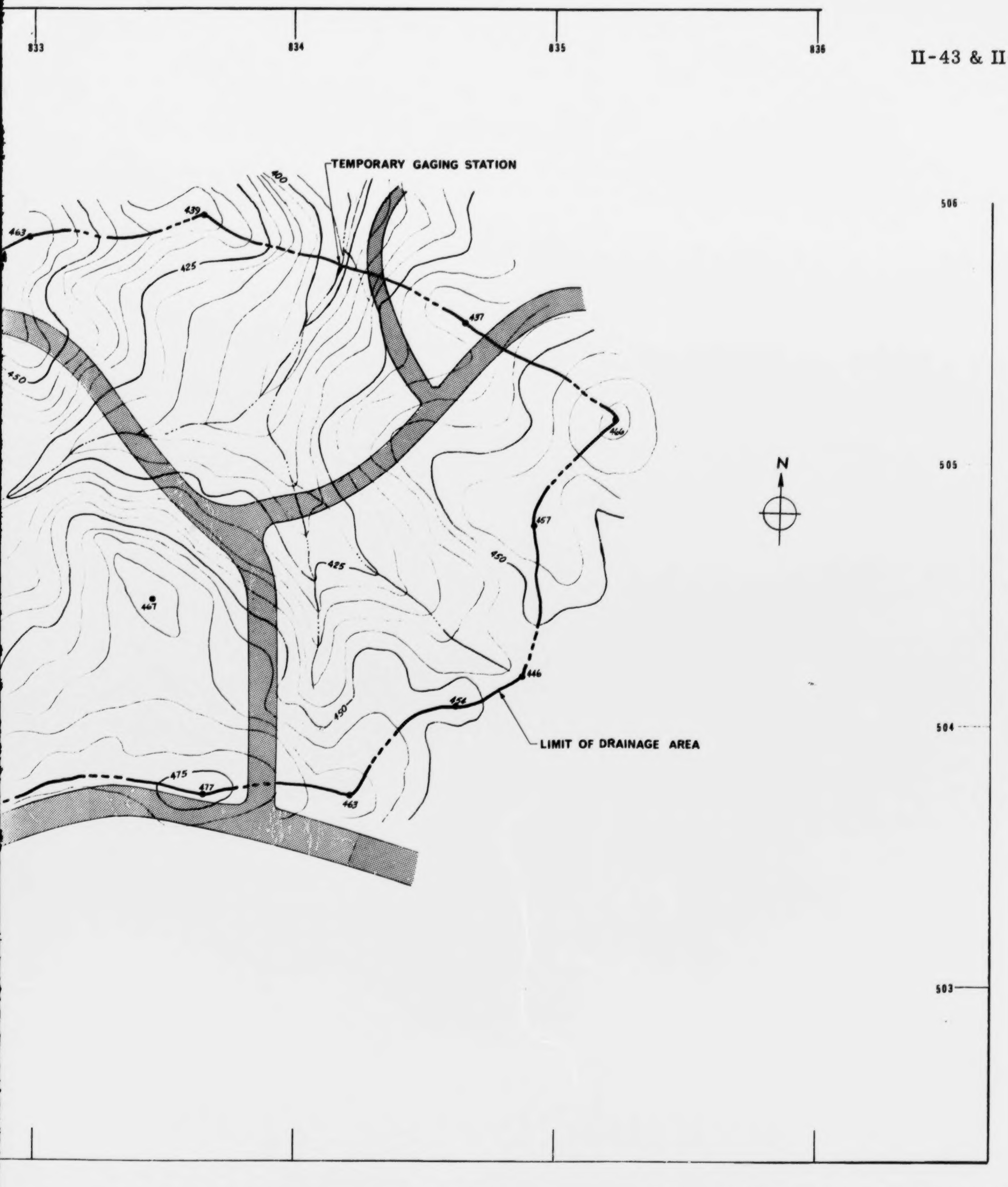
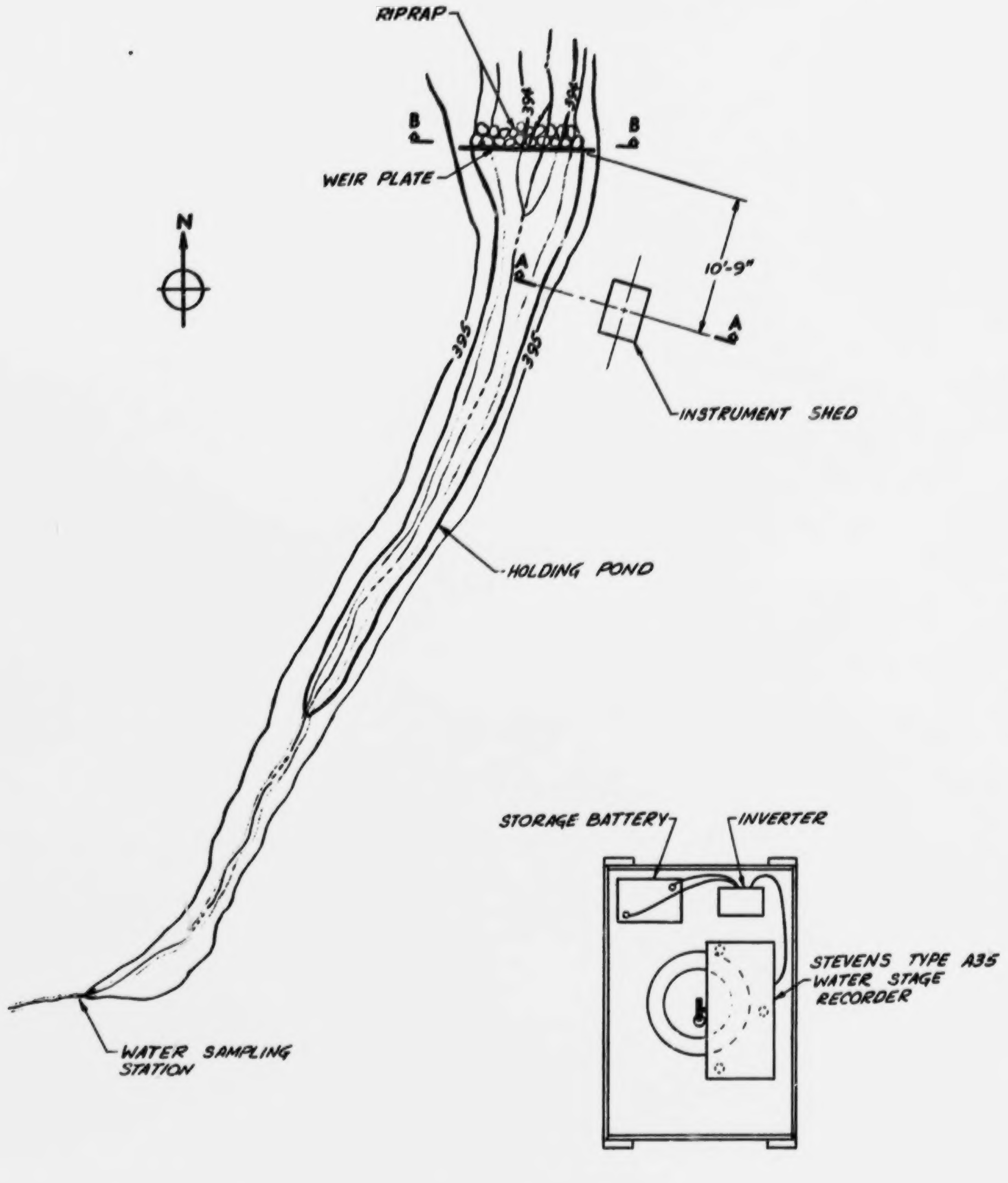
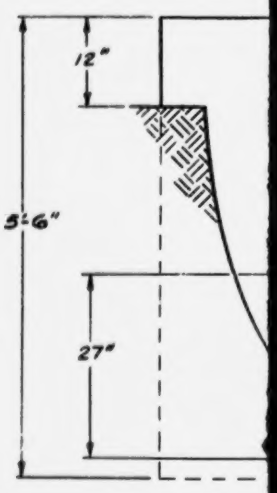

833
834
835
836
TEMPORARY GAGING STATION
400
463
439
425
437
450
466
506
505
504
503
N
457
450
425
467
446
456
450
LIMIT OF DRAINAGE AREA
475
477
463

Figure II-22. Temporary Stream Gaging Station Drainage Area

B

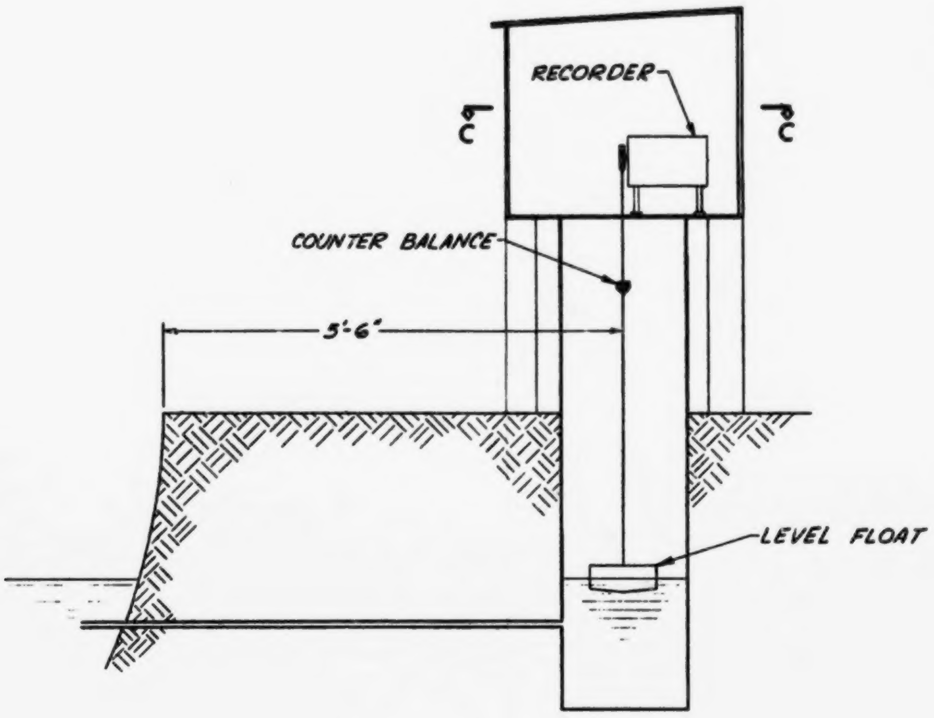

RIPRAP
394
394
B
B
WEIR PLATE
N
10'-9"
A
A
395
395
INSTRUMENT SHED
HOLDING POND
WATER SAMPLING STATION
STORAGE BATTERY
INVERTER
STEVENS TYPE A35 WATER STAGE RECORDER

SECTION C-C

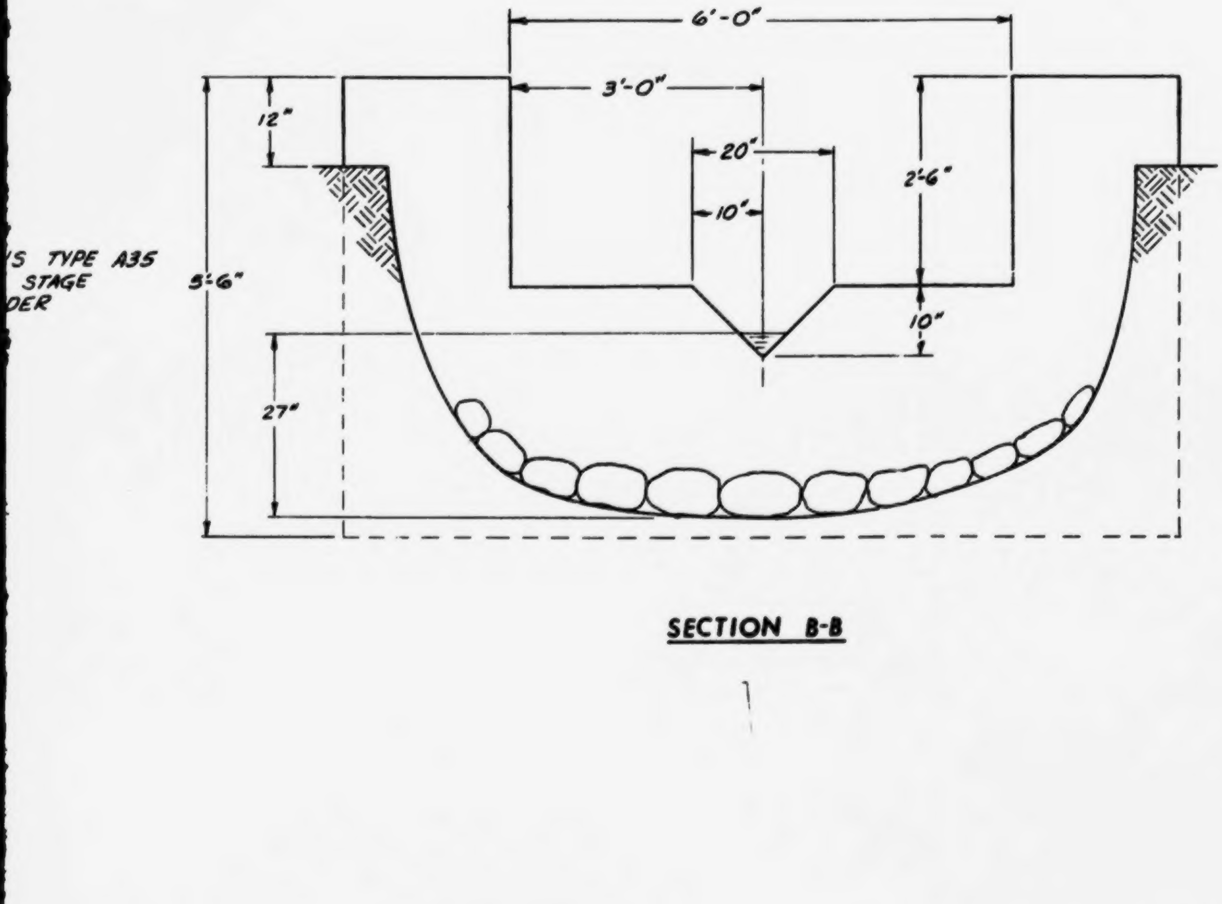


Hittman Associates, Inc.
[illegible]
[illegible]

A


RECORDER
C
C
COUNTER BALANCE
5'-6"
LEVEL FLOAT

SECTION A-A


6'-0"
3'-0"
20"
10"
12"
2'-6"
S TYPE A35
STAGE
DER
5'-6"
10"
27"

SECTION B-B

Figure II-23. Temporary Stream Gaging Station Site Plans

B

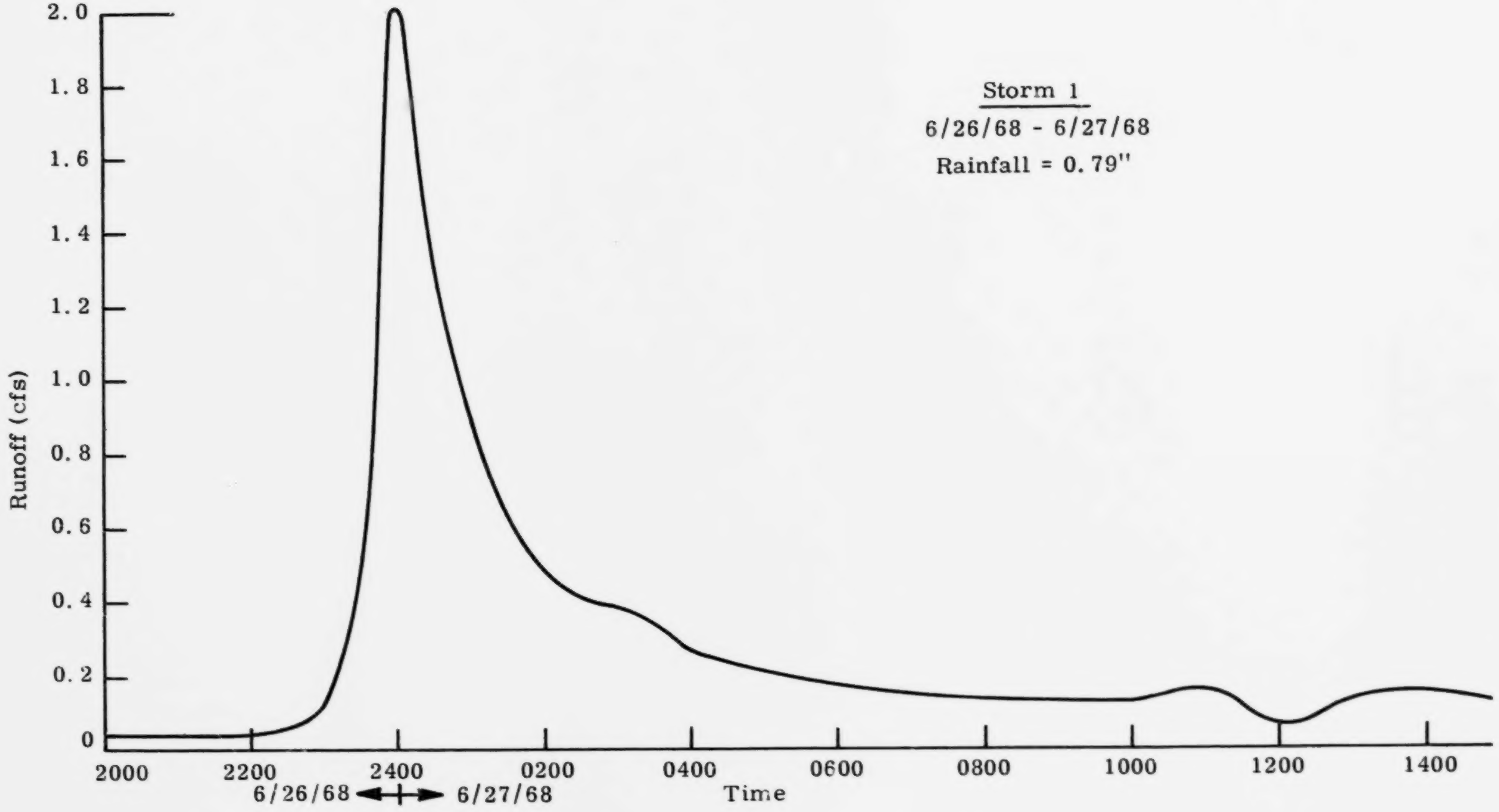


Figure II-24. Observed Hydrograph - Storm 1

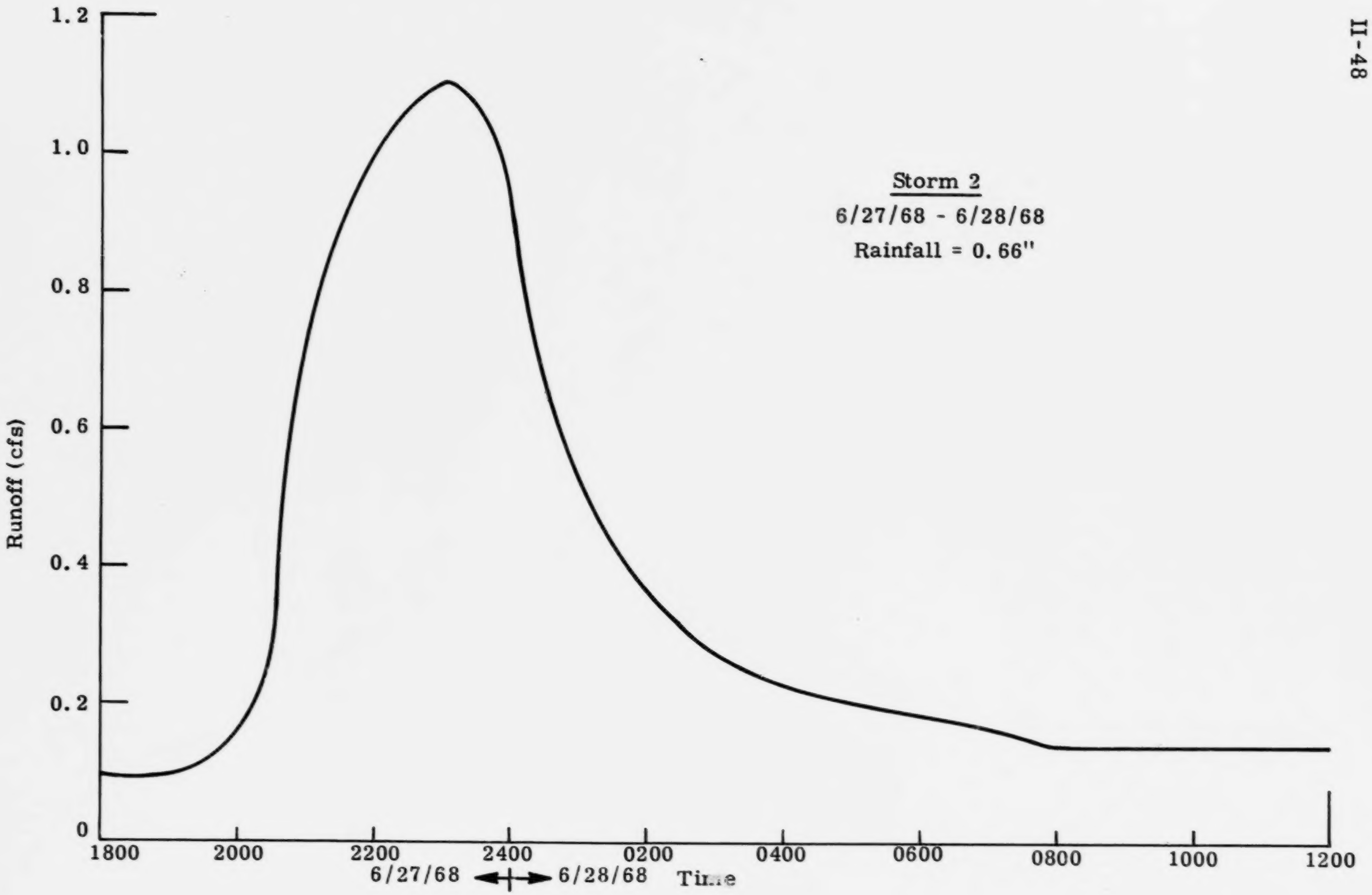


Figure II-25. Observed Hydrograph - Storm 2

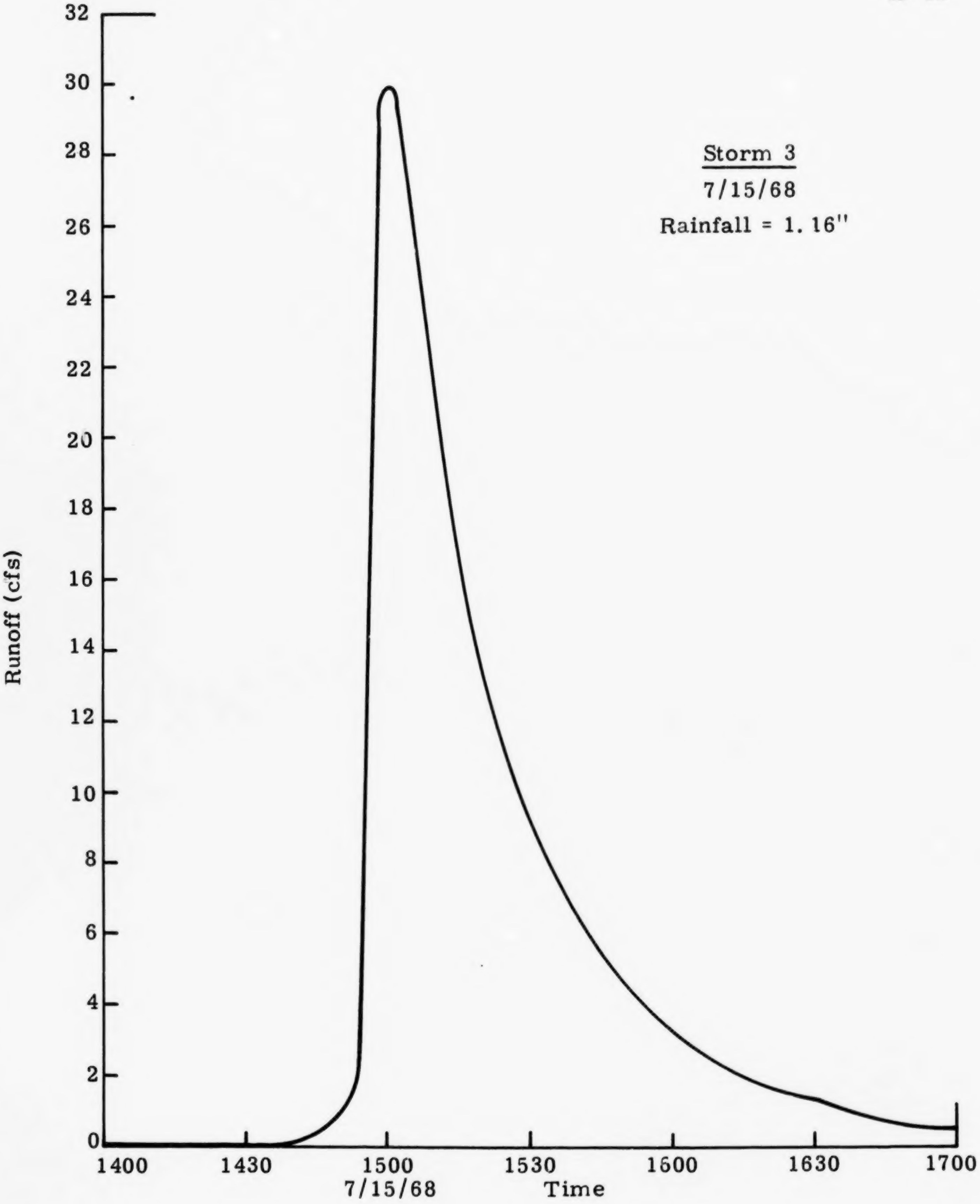


Figure II-26. Observed Hydrograph - Storm 3

D. Water Quality and Demands

1. Introduction

Storm runoff has been stored and reused locally within the catchment area for centuries. Many island communities, such as Gibraltar, Guam, and Bermuda, have been entirely dependent on local efforts to maximize this type of water resource to supply their needs. The Department of Agriculture publishes handbooks to assist farmers in constructing farm cisterns as supplementary water supplies. In contrast to water falling on clean and well-maintained, frequently impervious, catchments, the storm runoff from a typical urban area has acquired heavy pollutant loads before it enters a storm water collection system. In addition to discharging large quantities of pollutant into natural watercourses, storm runoff is sometimes permitted to enter combined sewer systems, or may accidentally enter sanitary sewers through leaks, improper connections, etc., where it causes overflows and further increases stream pollution.

The reuse of urban runoff as a supplementary water resource presents a number of problems, detailed elsewhere in this report. It is clear that the treatment of storm water to drinking water quality is potentially a difficult and expensive task. Section II.G discusses the treatment problem in some detail. Since many possible water uses do not require drinking water quality, an investigation was made of applications of sub-potable water within an urban community. A literature search was made to discover other applications of storm water to sub-potable reuse. No local reuse of storm water was reported in the literature surveyed, but numerous applications of sub-potable water systems, including reclaimed wastewater, were reported (Refs. 6, 7, 8, 10, 13, 20, 21, 22). Next, the kinds of reuse available in the Columbia area were investigated and listed. The various types of reuse were categorized into four water quality classes and criteria developed for estimating the total demand in each. Finally, demands were calculated for the Wilde Lake watershed and each of its component sub-watersheds.

2. Literature Search

A literature survey has been made of previously reported efforts to recover a wide variety of waste and sub-potable waters for distribution to uses that would otherwise be supplied by public potable water systems. This search was made to determine previous technical success with such projects, but, more particularly, to determine levels of demand for various types of reuse and to discover whether public acceptance of such systems was achieved. The largest portion of reported applications was devoted to use of reclaimed wastewater, normally from the municipal wastewater treatment plant. Although this involves a water resource somewhat more difficult to treat and probably much more unlikely to win widespread acceptance, published experience has been summarized to permit an examination of a more severe case than the one under study.

a. Wastewater Reclamation: Definition; History; Experience. The expression "wastewater reclamation" is used to overcome public aesthetic-- or, more properly, semantic -- objections to the term "sewage treatment"

(Ref. 6). In a broad sense, "wastewater reclamation" is defined as the purposeful upgrading of the quality of sewage, rendering it reusable by agriculture, industry or the public (Ref. 11). Planned wastewater reclamation has been practiced for over 30 years. All the basic use categories except drinking water supply are being directly served by reclaimed waters. Even though the present quantity of reclaimed wastewater being used is still rather small, planned reclamation appears to be on the threshold of becoming a major consideration in the augmentation of water resources (Ref. 9).

Wastewater reclamation falls into two categories: incidental and planned. Waste treatment plants discharging into fresh waters provide incidental reclamation in that the effluent may be used again. On the other hand, planned reclamation involves the production of water suitable for direct application to a beneficial use. Treatment facilities are financed and constructed specifically for this purpose (Ref. 9).

Reclamation of wastewater may be directed toward any of the following applications:

(1) Specialized industrial uses
(2) Crop and domestic irrigation
(3) Recreational lakes and ponds
(4) Ground water recharge
(5) Non-potable domestic uses, e.g., toilet flushing (Ref. 6)

A brief summary of reported experience in each of these areas of use follows:

(1) Industrial Uses. Direct use of treated municipal wastewaters by industry is presently small, but the increasing number of undertakings attest to the enormous potential that exists (Ref. 13). To date, most of the recirculation by industry has been limited to using industrial wastewater for cooling purposes, although some plants have started reusing process water, and a few have virtually closed systems requiring only small amounts of makeup water (Ref. 24).

Mining, metallurgical and allied industries located in arid regions of the United States have pioneered the reclamation of wastewater. Growing engineering knowledge and successful experience at older installations lead to the acceptance of water reclamation as more than just a novelty. Many large industrial plants find it desirable to treat and reuse wastewater because of significant financial savings (Ref. 10).

The literature reports that more than 150 industries located in 38 states are reclaiming industrial wastes. In addition, 15 industries in nine states employ treated municipal wastewater (Ref. 10). Two well-known examples are Bethlehem Steel Company's plant at Sparrow's Point, Maryland, which uses effluent from Baltimore's domestic sewage treatment plant, and the Kaiser Steel Company at Fontana, California, which reclaims industrial and domestic wastes from its own plant (Ref. 6).

The Bethlehem Steel Plant began using Baltimore sewage plant effluent in its steel mill in 1942. This proved to be a better water supply than the saline Chesapeake Bay or available ground waters which were increasing in salinity due to overdraft. Wastewater reclamation presently accounts for 22 percent of the plant's cooling water requirement (Ref. 13).

The quantities of water used by industry may often be radically reduced by proper reuse. The steel industry offers the best example of this. At the Kaiser Steel Plant at Fontana, the net use of water has been reduced to 1000 gallons per ton of steel. This is a substantial saving when compared to the national average of 65,000 gallons per ton of steel. Not only will the reuse of water affect the total quantity of water used, but reused water can often be purchased at a considerably lower cost (Ref. 23; see also Ref. 15-22).

(2) Agricultural Uses. Irrigation requires more water than any other use (Ref. 24). Methods of irrigation with reclaimed water include: flooding, spraying or sprinkling, ridge and furrow, and subsurface irrigation. Although most state health standards do not permit the use of sewage for truck farming regardless of the degree of treatment, treated effluent can be used on cotton, beets, vegetables grown for seed production, pasture crops, and woodlands (Ref. 13).

Crop irrigation has been used as an aid to disposal of community wastes, with irrigation being merely incidental to the disposal operation. "Sewage farms," a term which applies specifically to the agricultural utilization of sewage effluent, have been used to raise crops and pasture livestock (Ref. 6). In the process, they provide a degree of secondary treatment in addition to disposing of the water. At Fresno, California, the city sewage farm produces crops and beef. The farm also takes the place of a secondary treatment plant that would be needed to sufficiently clean the wastewater for disposal to a canal or stream. Additionally, a degree of treatment is provided for that portion of the water that filters through the soil to recharge ground waters (Ref. 10).

In dry desert areas, the agricultural utilization of effluents becomes a distinctly separate farming operation. The effluent is used in lieu of other irrigation water. Important limitations are the local requirements regarding the type of crops that can be raised (Ref. 10).

(3) Recreational Uses. One of the outstanding examples of recreational use of reclaimed wastewater is the installation at Golden Gate Park in San Francisco. This activated sludge plant is providing an effluent which is chlorinated and used for maintaining the level of decorative lakes and the irrigation of pastures, grass, and gardens. Some luxury Las Vegas hotels treat and use their own effluents to irrigate their grounds.

Irrigation of golf courses using reclaimed wastewater has increased in recent years. The Marine Bases at El Toro and Pendleton, California are examples. At El Cajon, near Palo Alto, California, there are semi-public courses now under construction which will use sewage effluent for irrigation. It has been shown that golf courses, employing sewage effluent, are able to maintain themselves without additional fertilization (Ref. 10).

A reclamation project has been undertaken by the Santee County Water District in San Diego County, California, which provides for the development of a series of artificial lakes fed by the effluent of a standard activated sludge sewage treatment plant. There are four lakes, two of which are open to the public for recreational uses. Playgrounds and picnic areas line the shores. Swimming is currently not permitted, and catching fish for human consumption is not allowed. However, a recreational area is provided, via use of reclaimed wastewater, in an area which affords little attraction of this type (Ref. 13).

It is believed that future application of reclaimed wastewater for irrigation of parks and golf courses holds considerable promise (Ref. 10).

(4) Ground Water Recharge Uses. Replenishment of ground waters through artificial recharge has been given considerable attention. In many areas, ground water levels have been rapidly falling and are not being restored through natural means. Annual water replacement may lag behind withdrawals due to the slow movement rate of ground water and limited opportunities for surface waters to penetrate the earth's surface. Waters suitable for artificial recharge may be classified generally as flood waters, industrial wastes, effluents, and municipal sewage (Ref. 13).

Numerous methods are used in artificial recharge operations. The oldest and still most common technique is the utilization of holding basins. Through construction of dikes and levees, a series of reservoirs are arranged to impound the water in such a manner that overflow from one will enter the next and so on. A second method is the use of a modified stream bed. A stream channel is widened and treated by a combination of methods to increase the percolation rate (Ref. 13).

Ditches and furrows may also be used. The basic arrangements are:

(a) Contour type - ditch follows ground contour

(b) Lateral type - water is diverted into a number of small furrows from a main canal

(c) Branching type - water is diverted from the primary channel into successively smaller ditches

Where slopes are relatively uniform, flooding provides an economical means of recharge. Normal practice is to spread the recharge water over the ground at small depths so as not to disturb the soil or native vegetation. Another recharge method is the pumping of water through injection wells directly into porous strata (Ref. 13 and 6).

The Whittier Narrows water reclamation project in Los Angeles County, California is an example of ground water recharge potential (Ref. 12). The object of this project is to conserve and assist in restoring the water resources of the agricultural area of Southern California. Treated sewage is distributed into spreading ponds. The reclaimed waters than percolate through to the ground water aquifers and are eventually used in the irrigation of agricultural lands. Operation of the Whittier Narrows plant is being closely watched by various agencies interested in public health, education, pollution control and industry (Ref. 13).

Ground water recharge by treated wastewater is an incidental, unplanned occurrence in numerous locations throughout the United States. It now appears that planned reclamation of wastewater for recharging underground basins will be practiced more and more extensively as knowledge continues to develop (Ref. 10).

(5) Domestic Uses. Domestic uses of reclaimed wastewater are generally limited to lawn watering and toilet flushing with a few notable exceptions. These exceptions are relatively recent developments. Some of the domestic uses reported are:

(a) Sites of Grand Canyon National Park, where treated domestic wastewater was first used in the park in 1926 for toilet flushing, lawn sprinkling, cooling water, and boiler feed water at the power plant

(b) Pomona, California, where municipal sewage plant effluent has been used for domestic irrigation of lawns and gardens in a suburban home development since 1929

(c) San Diego State Teachers College, California, where sewage effluent was first used for lawn and shrubbery irrigation in 1931

(d) Golden Gate Park, San Francisco, California, which was mentioned earlier, first used activated sludge plant effluent in 1932 for lawn watering and maintaining water levels in some of the park lakes (Ref. 6)

A pilot plant in Tucson, Arizona has demonstrated that potable water may be made from municipal waste treatment effluent. The process is automatic and uses effluent from an activated sludge, process, followed by softening, flocculation, filtration, demineralization, and carbon absorption (Ref. 14).

A more striking example is the experience of Chanute, Kansas. In 1956, when drought dried up the local water supply, effluent from the city's secondary treatment plant was diverted to the water supply reservoir. Treated wastewater was recycled through the city eight to fifteen times during a five month period under the supervision of public health authorities (Ref. 8 and 14). It is interesting to note that in the Chanute case, the municipality itself made the decision to reuse its sewage as the best possible solution to the water supply problem (Ref. 8).

All of the examples in this section indicate that reuse of wastewater is not only feasible but has been successfully practiced in several areas. It has been used for industry, agriculture, and recreation as well as for ground water recharge and domestic uses. Although domestic use of reclaimed wastewater is a relatively new concept, it appears to have considerable promise and growth potential.

b. Dual Systems: Definition; History; Experience. For situations where domestic water requirements can be partially met by a nearby source of reasonably clean water, a dual water system may be used. Dual water systems can be defined as those involving distribution of two grades of water to consumers through independent pipe networks (Ref. 7).

The idea of dual water systems for domestic supply received consideration in the United States as early as 1894. At first, universal adoption of the concept was not advocated. It was urged, however:

> "Where there is a supply of naturally pure water sufficient to meet culinary and drinking requirements, it [should] be supplied through a separate system, leaving the larger requirements to be furnished from the nearest, cheapest source of reasonably clean water" (Ref. 7).

The water of Coalinga, a city of 6, 300 located in South Central California, is highly mineralized and unsatisfactory for drinking purposes and most home uses. Because of the poor quality of this supply, softer water is shipped to Coalinga from Armona, California by railroad car. Since 1931, this water has been distributed to the homes through a system constructed in parallel with the hard water system. Both supplies are safe for drinking purposes. Thus, Coalinga has been operating a dual water system for over 35 years (Ref. 7).

Avalon, a town on Catalina Island offshore from Los Angeles, operates a dual water system. This system has been in operation since 1914. A fresh water distribution system supplies water for drinking, cooking, and bathing. A salt water system supplies ocean water for fire protection and sanitary purposes. It appears that the operation of this system will continue due to the shortage of fresh water on the island (Ref. 7).

Another example of a dual water system is the one in operation at the University of Wisconsin since sometime prior to 1913. The University draws water from Lake Mendota which is treated and pumped to the buildings and used for showers, laboratories, air conditioning, fire protection, boilers and other uses not involving human consumption. Drinking water is purchased from the city of Madison and supplied to the same buildings through a separate distribution system (Ref. 7).

For public health reasons, community wide distribution of biologically safe and unsafe waters through a dual system cannot be seriously considered. If the question of dual water systems is examined on the condition that both supplies are safe for drinking purposes, then the concept appears worthy of study. The two supplies are usually termed "potable" and "sub-potable" (Ref. 7).

The way in which water is used within the home is a matter of importance in evaluating dual systems. Some figures published recently suggests the following percentages of total home uses as shown in Table II-6.

TABLE II-6. DISTRIBUTION OF RESIDENTIAL WATER USE, AFTER REID (Ref. 25)

	% of Total	Gal. /Capita/Day
Toilet flush	28	24.0
Bathing	23	20.0
Dishwashing	4	3.75
Drinking and other kitchen	3	2.75
Laundry	10	8.5
Lawn sprinkling	29	25.0
Auto washing	3	2.5

These percentage figures are only approximate, but they do portray the high proportion of water used for purposes other than human consumption.

The potable supply is necessarily of excellent quality, suitable in all respects for drinking, cooking, and bathing. It is used for dishwashing, cleaning, and operation of garbage disposal units. (Home garbage disposal units are presently an integral part of the kitchen sink, thus it is not feasible to provide water from a sub-potable supply). Potable uses amount to an average of 26 gallons per capita per day (Ref. 26).

The ideal potable supply should be uniformly clear, colorless, tasteless, odorless, low in dissolved minerals, and free from harmful biologic forms. The water should not be corrosive or encrusting at normal temperatures or at higher cooking temperatures. If desired, fluoride ion levels may be adjusted for prevention of dental caries (Ref. 7).

Since potable uses account for no more than 30 percent of the total home needs, supplies of good quality water that would be inadequate for conventional systems may be suitable for development of a dual water system. Thus, the employment of a dual water system offers an opportunity to conserve supplies of good quality water.

The sub-potable supply, inferior in chemical quality but equal in safety, can be used for waste transport, lawn and garden irrigation, street flushing, fire protection, and commercial and industrial uses that do not require high quality water. The sub-potable supply should be given treatment that will render it safe, non-staining, and not excessively corrosive or scale forming.

The sub-potable supply should be maintained at a quality level such that occasional inadvertent use for drinking purposes will not result in harm to the consumer. The supply must be free from harmful biologic forms and toxic chemicals. Elements of the sub-potable system that furnish water for normal and fire fighting demands must provide hydraulic capacity for fire flows plus the coincident draft for toilet flushing, irrigation, and other domestic and industrial uses.

c. Problems Affecting Public Acceptance. The literature is essentially silent on public reaction to the aesthetics of wastewater reclamation and dual water systems. It can be observed, however, that the public has accepted existing projects. Even though this acceptance has occurred, the aesthetics of reclamation is still considered an important factor (Ref. 9).

Hundreds of U.S. cities draw their water from inland streams. They are, in fact, using the diluted sewage effluent of upstream cities. Although the presence of such pollution is generally known, consumer acceptance of the water does not appear to be appreciably reduced. Dilution and time appear to dissipate aesthetic objections. An assumed public aversion to the concept of direct wastewater reclamation has been a major deterrent to expansion of this practice (Ref. 9).

When the public is aware that reclamation is taking place, the developers of the project must create public confidence. Such confidence is merited only when the projects are developed along sound engineering lines (Ref. 6). The previously mentioned Whittier Narrows plant has been designed for and encourages routine public inspection. It is pleasingly landscaped, operation is odorless and nuisance-free, and high quality water is produced (Ref. 9).

It was mentioned earlier that in many cities, the municipal water intake is only a short distance downstream from the point of sewage discharge of another city. Some cities prefer to transport good water long distances rather than treat polluted water available nearby. Examples of these two philosophies are Philadelphia and New York City. Both have large, notoriously polluted rivers running past them. Philadelphia treats the water from the nearby Schulkyll and Delaware Rivers, while New York builds dams and transports clean water more than 100 miles through a series of reservoirs and aqueducts (Ref. 24). Both cities produce water of excellent quality.

When dual water systems are mentioned, objections regarding health hazards are raised. These objections arise from associating the word "dual" with the hazardous combination of potable and contaminated water (Ref. 7). Early in the development of dual water systems, it was established that the most serious difficulty in operating such systems "will be found in the proper regulation of the use of purified water," (Ref. 7). It was felt that some customers would use the pure water for other than intended uses, while others, either carelessly or to avoid cost, would not use it at all. To control this difficulty, a two-step solution was suggested. First, meters could be used on both water systems and, second, a municipal ordinance, enforced by sanitary inspectors, requiring the use of the better water for certain stipulated purposes could be passed (Ref. 7). Hopefully, these suggestions, if put into use, would not only control the use of the better water, but would prevent human consumption of the contaminated supply.

Conversely, the two water supplies in existing dual water systems are both safe for drinking purposes. The "potable" supply is maintained at an excellent quality, suitable in all respects for drinking, cooking, and bathing. The "sub-potable" supply, inferior in chemical quality but equal in safety, is suitable for waste transport, lawn and garden irrigation, street flushing, fire protection, and other uses not requiring water of high quality. Each dwelling served by a dual water system requires two service lines and two meters (Ref. 7). Because each water supply is biologically safe, a municipal ordinance such as that suggested earlier has not proven necessary.

The situation is not quite the same when industry uses reclaimed water. Experiences from the past have demonstrated that water from secondary supplies is sometimes used unwittingly by workers. As this water is not of the same biological standard as the secondary water supply for domestic use, suitable means must be devised to prevent human consumption. Differential marking of pipes, adequate marking of the outlets, and possibly the use of special control valves that can be operated only by certain personnel are some methods that may be used to deal with this problem. Employee education is a further means of meeting the situation (Ref. 6). The problems associated with the use of reclaimed wastewater in industry are not insurmountable as evidenced by the successful operation of the Bethlehem and Kaiser steel plants.

The difference between treatment systems for water reuse and for disposal to streams seems to be one of degree rather than of kind. At least no sharp distinction is evident in the literature. Streams and lakes are natural systems for bio-treatment. Their water is often reused several times before discharge to the sea. Present public acceptance of "natural" processes is considered greater than acceptance of engineering processes (Ref. 4).

3. Reuse Classifications

Storm water was considered as a potential water source for every level of water use existing within the Wilde Lake area. Although each discrete use has its own unique water quality requirement or standard, an attempt will be made to draw the various uses into a limited number of categories, simplifying further analysis. Whenever a group of use types are defined as one category, the water quality standard selected for that category must meet the requirements of each use considered.

Water use can be broadly described as being either residential, commercial, industrial, or public. Table II-6 presented earlier, listed various types of residential water uses, with approximate percentages of total consumption. Linaweaver, et al., collected data from more than 3000 residences in public metered water and public sewer areas of the eastern United States and analyzed that data in terms of residential consumption not including sprinkling use and sprinkling use alone (Ref. 26). All areas studied in the eastern U. S. averaged 56. 7 gallons per capita per day for non-sprinkling residential use. The thirteen local areas studied ranged from 42. 9 to 71. 1 gallons per capita per day (Ref. 27). Examination of similarities between areas studied by Linaweaver and the Wilde Lake region of Columbia led to the selection of 61. 5 gallons per capita per day as an estimate of water consumption within the dwelling unit.

Sprinkling use was conclusively correlated, in the Linaweaver work, to the irrigable area associated with the dwelling unit and the local deficiency between potential evapotranspiration and summer precipitation (Ref. 28). Three areas were located within the immediate vicinity of Columbia for which data was available, known as Glenmont, English Manor, and Northwest Branch Estates. All are served by the Washington Suburban Sanitary Commission and are comprised of homes similar in value and lot size to those in Columbia. Average annual sprinkling use, as computed by Linaweaver, divided by average irrigable area for each of these locations resulted in sprinkling usages ranging from 253 gallons per day per acre to 921 gallons per day per acre (Ref. 27). In order to arrive at a value for estimating demand in Columbia, these three areas were averaged, and the mean was found to be 518 gallons per day per acre. Table II-6 was revised, with slight regrouping of uses, due to certain uses sharing common plumbing fixtures, making separation unfeasible. The revised table is shown as Table II-7.

TABLE II-7. DISTRIBUTION OF RESIDENTIAL WATER USE - REVISED

Quality	Use	Gal/ Capita/Day	Gal/ Acre/Day
AA	Human consumption, food preparation, general kitchen use	6.5	
A	Bathing, laundering, auto washing	31.0	
B	Lawn sprinkling		518.0
C	Toilet flushing	24.0	

The water quality classification shown in Table II-7 have been assigned in decreasing order of quality level. Water quality class "AA" is defined as meeting the U. S. Public Health Service Drinking Water Standards and is intended for all potable water uses (Ref. 29). Class "A" is virtually identical to "AA" except for taste and odor considerations not important in water which will not actually be consumed, or used in food preparation. Class "B" has somewhat more lenient requirements, particularly with respect to suspended solids, except that it must be disinfected to the same standard as Class "AA", thus providing safety for accidental ingestion. Class "C", also disinfected, has little more than a suspended solids limit and minimal requirements for corrosivity. Specific limits on various quality parameters are shown in Table II-8. This assignment of water quality levels to the various components of residential use is generally consistent with the independent analysis carried out by Whitman, Requardt and Associates, and reported as Appendix B.

Commercial water use might contain all of the use categories discussed above, in addition to air conditioning loads and other miscellaneous commercial uses. In all cases, the water demands of each commercial user must be evaluated with respect to the four water quality levels described above and defined in Table II-8. Particular types of commercial use vary widely

TABLE II-8. MAXIMUM CONCENTRATION OF SELECTED POLLUTANTS BY REUSE CATEGORY

Constituent (mg/l unless indicated)	Required Water Quality* Maximum Concentration			
	AA	A	B	C
Alkyl benzene sulfonate	0.5	0.5	0.5	0.5
Ammonia (NH_3)	0.5	0.5	0.5	0.5
Arsenic	0.01	0.05	0.05	0.05
Barium	1.0	1.0	1.0	1.0
Cadmium	0.01	0.01	0.01	0.01
Calcium	75	75	75	75
Carbon chloroform extract	0.2	0.2	0.2	0.2
Chloride	250	250	250	250
Chromium (hexavalent)	0.05	0.05	0.05	0.05
Copper	1.0	1.0	1.5	1.5
Cyanide	0.01	0.2	0.2	0.2
Fluoride	1.5	3.0	3.0	3.0
Iron	0.3	0.3	0.3	0.3
Lead	0.05	0.1	0.1	0.1
Magnesium	50	150	150	150
Magnesium+Sodium sulfate	500	500	1000	1000
Manganese	0.05	0.1	0.5	0.5
Nitrate (as NO_3)	45	50	50	50
Oxygen, dissolved (minimum)	5.0	5.0	4.0	4.0
Phenolic compounds (as phenols)	0.001	0.002	0.002	0.002
Selenium	0.01	0.05	0.05	0.05
Silver	0.05	0.05	0.05	0.05
Sulfate	250	250	400	400
Total solids	500	500	500	500
Zinc	5.0	15	15	15
BOD (5-day) mg/l	-	-	-	-
Coliform (MPN per 100 ml)	1	70	240	240
pH (units)	7.0	6.0	6.0	6.0
Color (units)	15	20	30	30
Turbidity (units)	0-3	3-8	8-15	15-20
Suspended solids	-	-	10**	30**
Phosphates	1.0	1.0	1.0	1.0
Volatile suspended solids	-	-	-	-

*Based upon maximum concentrations allowed by the USPHS, The World Health Organization, and the Water Quality Standards of the State of Md.

**Higher suspended solids are permitted by various water quality standards. Limit based on sediment control and water contact recreation.

not only in the specific uses involved, but in the relative magnitude of each of them.

Industrial uses fall, to a great degree, into the same type of breakdown as commercial uses. Additionally, industry frequently has large demands for process water, whose required quality varies from very low to substantially higher than potable water quality, depending on the industry and the process employed. In order to compute demands for each water quality, a particular industry must be analyzed in detail to determine its requirements by quality level. In the study area, the Wilde Lake watershed of Columbia, no process-water-using industry is contemplated, making this type of analysis unnecessary in the present study.

Public water uses include applications such as street cleaning, sewer flushing, fire protection, and snow melting. Whitman, Requardt, and Associates have given estimated consumption for each of these uses in Appendix B. Street cleaning and sewer flushing can be seen to account for very little water use on a long-term basis, compared to the available supply. Providing water for fire protection implies that high flows will be available from a system of very high reliability. Fire insurance rating bureaus normally insist that the required fire flows for a community like Columbia be available entirely from elevated storage, independent of pumping capacity. The lowest requirement in the Wilde Lake watershed applies to the detached single family residential areas, where 120,000 gallons of storage must be continuously available. In order to consider a number of separate systems, each of them must have the full fire flow capability. Furthermore, the area under study is already served by the Howard County Metropolitan Commission with a conventional public water supply system designed to provide necessary fire flows. For these reasons, fire flows were not considered as possible uses of the storm water within the local reuse concept. Another public use considered was the flushing of streets following snowfalls to melt and remove accumulated snow. This is nonconsumptive use from the standpoint of the storm water system, since the runoff returns to the storage facility. This type of snow removal is confined to those occasions when pavement temperature is above 40°F and air temperature is at least 30°F, to prevent re-freezing of the water as ice. The flushing must also be done at a time when the pavement will have the opportunity of drying prior to the next freezing cycle, effectively preventing use of the technique at any time except under bright sunlight. Unfortunately, the circumstances which permit the use of water flushing of snow usually result in rapid melting of the snow without flushing. The water required on the infrequent occasions when this procedure is indicated is considered to be a negligible amount.

4. Distribution of Water Demands Throughout Watershed

In order to permit evaluation of various levels of treatment for each of the sub-watersheds and combinations of sub-watersheds, the water demands were tabulated, by water quality class, for each sub-watershed. The boundaries of the sub-watersheds were revised slightly for this tabulation in order that the area of computation be similar to that which would actually be served by a discrete distribution system within the sub-watershed.

Figure II-27 shows the distribution areas, divided by broken lines, except for the sub-watersheds lying wholly outside the Columbia project, where the natural drainage lines will continue to define the areas. A second tabulation was made of land uses within the revised sub-watersheds, analagous to that shown previously as Figure II-20. This tabulation, shown as Figure II-28, also lists irrigable area, defined as the difference between total area and impervious area, excepting open space. This is the approximate area of all lawns, gardens, and school grounds which are likely to be irrigated by lawn sprinklers during the summer.

The usage coefficients described earlier for each of the water quality classes are applied to the appropriate parameters and residential water use is computed for each quality level. Other demands, including commercial and institutional uses, are estimated individually for each installation and listed on the sheet. Residential and other demands can be totalled to yield water demand by water quality class for the sub-watershed. This operation was performed on each sub-watershed in the entire drainage area to evaluate total demands. Prior to assembling this information for input to the systems analysis computer program, however, the demands listed for class "B" uses, lawn sprinkling, were derated. In a later section, storage requirements associated with several patterns of water demand were evaluated. The extremely non-uniform behavior of sprinkling demands was found to place a significant additional requirement on reservoir size, unless a portion of that demand could be satisfied by other means. Using the methods described in Section II-F, an analysis was performed to determine the level of reliability (defined as percent of demand satisfied by the storm water reuse system) with which sprinkling demands could be met without imposing excessive storage requirements on the system. It was determined that when sprinkling demands were satisfied with an 87.5 percent reliability and the balance of the demand with a 100 percent reliability, the reservoir requirement was essentially independent of the fraction of total demand which was composed of sprinkling uses. In other words, sprinkling demands are interchangeable with other demands, provided no more than 87.5 percent of the sprinkling use is satisfied by the system. The balance of the demands would be supplied from the public water system, through a make-up water connection. As a result of this analysis, sprinkling demands in the watershed were reduced to 87.5 percent of their original value, then treated on the same basis as other, uniform demands with respect to storage requirements.

Examination of the water use categories indicates that eight possible combinations of water use exist. This includes all possible combinations of the lower three quality classes, as well as all four classes together. It was assumed that, should potable water be produced, there would be no incentive for separating plumbing systems, therefore all demands would be connected to a single system. In all the other cases, where sub-potable water is being produced, the plumbing systems constructed for sub-potable water distribution might be connected to any combination of fixtures.

The water demands computed for each sub-watershed, after derating of the class "B" demand as discussed above, were combined into each of the possible use combinations and displayed on a table. This table was input to the computer program to serve as the basis for evaluating various alternate reuse systems. A printout of this table appears as Figure II-29.

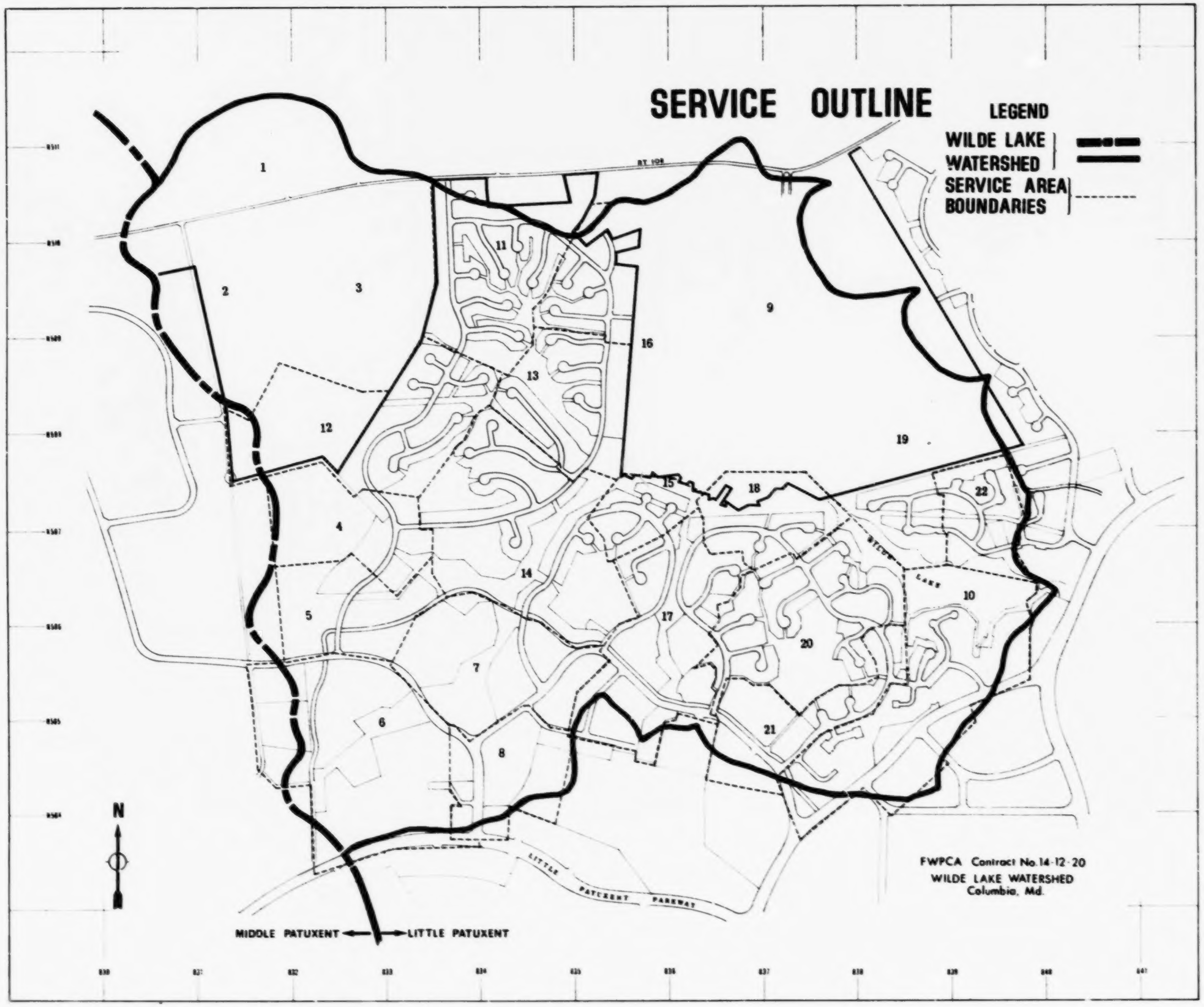
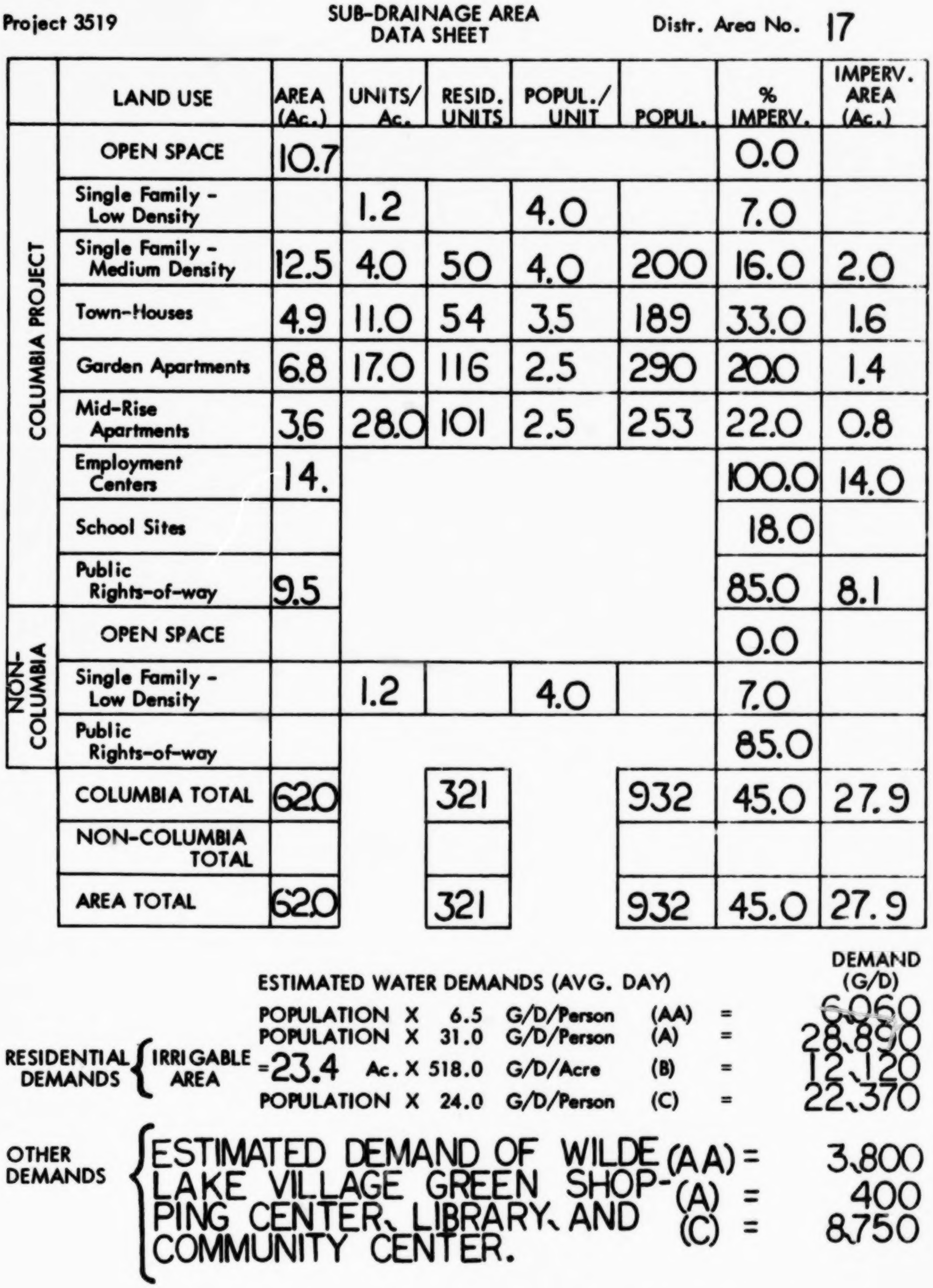

SERVICE OUTLINE
LEGEND
WILDE LAKE WATERSHED
SERVICE AREA BOUNDARIES
RT. 108
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
WILDE LAKE
LITTLE PATUXENT PARKWAY
N
MIDDLE PATUXENT
LITTLE PATUXENT
FWPCA Contract No. 14-12-20
WILDE LAKE WATERSHED
Columbia, Md.

Figure II-27. Wilde Lake Watershed - Service Areas



Project 3519

SUB-DRAINAGE AREA DATA SHEET

Distr. Area No. 17

	LAND USE	AREA (Ac.)	UNITS/ Ac.	RESID. UNITS	POPUL./ UNIT	POPUL.	% IMPERV.	IMPERV. AREA (Ac.)
COLUMBIA PROJECT	OPEN SPACE	10.7					0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Single Family - Medium Density	12.5	4.0	50	4.0	200	16.0	2.0
	Town-Houses	4.9	11.0	54	3.5	189	33.0	1.6
	Garden Apartments	6.8	17.0	116	2.5	290	20.0	1.4
	Mid-Rise Apartments	3.6	28.0	101	2.5	253	22.0	0.8
	Employment Centers	14.					100.0	14.0
	School Sites						18.0	
	Public Rights-of-way	9.5					85.0	8.1
NON-COLUMBIA	OPEN SPACE						0.0	
	Single Family - Low Density		1.2		4.0		7.0	
	Public Rights-of-way						85.0	
	COLUMBIA TOTAL	62.0		321		932	45.0	27.9
	NON-COLUMBIA TOTAL							
	AREA TOTAL	62.0		321		932	45.0	27.9

ESTIMATED WATER DEMANDS (AVG. DAY)

				DEMAND (G/D)
RESIDENTIAL DEMANDS	POPULATION X 6.5 G/D/Person	(AA)	=	6,060
	POPULATION X 31.0 G/D/Person	(A)	=	28,890
	IRRIGABLE AREA =23.4 Ac. X 518.0 G/D/Acre	(B)	=	12,120
	POPULATION X 24.0 G/D/Person	(C)	=	22,370

OTHER DEMANDS	ESTIMATED DEMAND OF WILDE LAKE VILLAGE GREEN SHOPPING CENTER, LIBRARY, AND COMMUNITY CENTER.	(AA)	=	3,800
		(A)	=	400
		(C)	=	8,750

Figure II-28. Work Sheet - Typical Sub-Watershed Land Use Tabulation and Water Demand Calculation

```
                                                  **** OUTPUT ****

                                        **** WATER DEMANDS BY SERVICE LEVEL ****

WATERSHED        AA          A           B           C          A,B         A,C        A,B,C        B,C       AA,A,B,C

    1          550.0      2600.0      7210.0      2020.0      9810.0      4620.0     11830.0      9230.0     12380.0
    2          550.0      2600.0      7298.0      2020.0      9898.0      4620.0     11918.0      9318.0     12468.0
    3          210.0       990.0      3120.0       770.0      4110.0      1760.0      4880.0      3890.0      5090.0
    4         1960.0      9180.0      6799.0      7480.0     15979.0     16660.0     23459.0     14279.0     25419.0
    5         2550.0     12150.0      4078.0      9410.0     16228.0     21560.0     25638.0     13488.0     28188.0
    6        17310.0     82550.0     20983.0     63910.0    103533.0    146460.0    167443.0     84893.0    184753.0
    7         9080.0     43310.0     11830.0     33530.0     55140.0     76840.0     88670.0     45360.0     97750.0
    8         4050.0     18230.0      7980.0     15060.0     26210.0     33290.0     41270.0     23040.0     45320.0
    9         2730.0     13000.0     36930.0     10100.0     49930.0     23100.0     60030.0     47030.0     62760.0
   10        45460.0     33670.0      9520.0     34760.0     43190.0     68430.0     77950.0     44280.0    123410.0
   11         3150.0     15000.0     12100.0     11620.0     27100.0     26620.0     38720.0     23720.0     41870.0
   12         1480.0      7070.0      5800.0      5470.0     12870.0     12540.0     18340.0     11270.0     19820.0
   13         2340.0     11160.0     10330.0      8640.0     21490.0     19800.0     30130.0     18970.0     32470.0
   14         5280.0     23500.0     15910.0     21690.0     39410.0     45190.0     61100.0     37600.0     66380.0
   15          810.0      3840.0      3130.0      2980.0      6970.0      6820.0      9950.0      6110.0     10760.0
   16         2310.0     11040.0     21480.0      8540.0     32520.0     19580.0     41060.0     30020.0     43370.0
   17         6060.0     28890.0     10610.0     22370.0     39500.0     51260.0     61870.0     32980.0     67930.0
   18         1200.0      5700.0      5710.0      4420.0     11410.0     10120.0     15830.0     10130.0     17030.0
   19         1140.0      5460.0     12190.0      4220.0     17650.0      9680.0     21870.0     16410.0     23010.0
   20         3460.0     17800.0     17590.0     16280.0     35390.0     34080.0     51670.0     33870.0     55130.0
   21         5430.0     25890.0     10560.0     20040.0     36450.0     45930.0     56490.0     30600.0     61920.0
   22         1460.0      6940.0      6210.0      5380.0     13150.0     12320.0     18530.0     11590.0     19990.0
```

Figure II-29. Summary of Water Demands
(Gal. per Day)

E. Storm Water Quality

1. Urban Runoff Quality

a. Literature Search. Section II.A described the requirement for a well-defined effluent quality standard to permit determination of the achievement of the pollution control objective of the local reuse system. In order to specify the various treatment alternatives which might be considered in the development of the optimum system to achieve that objective, it is necessary that the pollutional content of the storm water be known to some reasonable degree. At the least, the nature and approximate maximum levels of concentration of the principal pollutants should be known. The first attempt towards determining these parameters was to search published reports for analysis of urban runoff from similar areas. Numerous references to storm water quality were located, but only a few went further than isolated, unrelated pieces of data (Refs. 30 - 38). Among the several extensive studies of storm water pollution, most failed to relate specific pollutants to conditions in the watershed such as land use, industry, industrial and commercial wastes, etc. Pollutant concentrations quoted seldom included any reference to the intensity of runoff at the time of sampling, or the hydrograph previous to the sampling time. One of the more complete studies of storm water pollution was performed in Cincinatti, Ohio, by Weibel, et al, yet it did little more than set up some broad ranges of pollutant concentrations for watersheds similar to the one analyzed (Ref. 31). Table II-9 summarizes the observed range of a number of pollutants found in storm water at a variety of locations throughout the world. Each of the studies used to develop this table were sufficiently comprehensive to exclude the possibility of a chance or "freak" occurrence of an unrepresentative concentration of some specific pollutant. It can be seen that many of the items vary by two or more orders of magnitude throughout reported experience. It was concluded that the literature survey would be of little value in establishing a pollutant baseline for the Columbia location, due to the very wide variance of reported data. The sections below describe the tentative listing of maximum concentrations for expected pollutants, followed by the results of the water quality sampling station, intended to verify the tentative data.

b. Tentative Description of Runoff Quality. A review of the specific pollutants mentioned in various studies led to a classification of the parameters as shown in Table II-10.

(1) Solids. None of the studies reported above contained any quantitative data on floating solids. Inlet structures are normally expected to exclude large floating objects (boards, tree branches, toys, etc.) but the existence of open drainage channels downstream from the closed storm water collection system permits such objects to enter the flow. In addition, leaves, small branches, paper or cardboard objects, etc., are commonly found in all storm water collection systems. Much of the debris will eventually absorb water and sink, joining suspended solids, but some will remain bouyant until it is manually or mechanically removed from the stream. To

TABLE II-9. REPORTED STORM WATER POLLUTANT CONCENTRATIONS*

	Observed Range	Number Locations
Dissolved Solids	30 - 8,000	1
	154 - 228	1
Suspended Solids	26 - 36,250	6
Volatile	38 - 98	1
Nonvolatile	119 - 292	1
BOD	6.9 - 625	9
COD	18 - 3,100	2
DO	6.4 - 8.0	1
Total Nitrogen (NO_2, NO_3, Organic)	2.3 - 11.8	3
Total Phosphate	0.47 - 1,400	2
Coliform Total/100 ml	40 - 240,000	4
Fecal Strep/100 m.	median - 20,500	
Chlorides	11 - 160	2

*Refs. 30, 31, 34, 35, 36, 37, 38

TABLE II-10. PRINCIPAL INDICES OF STORM WATER POLLUTION

Type	Pollutant	Unit of Measure
Solids	Floating Solids	lb/day
	Suspended Solids	mg/l
	Dissolved Solids	mg/l
Oxygen Balance	Biochemical Oxygen Demand (BOD)	mg/l (5-day)
	Chemical Oxygen Demand (COD)	mg/l
	Dissolved Oxygen (DO)	mg/l
Nutrients	Total Nitrogen (N, NO_2, NO_3, etc.)	mg/l
	Total Phosphorous (P, PO_4, etc.)	mg/l
Bacteria	Coliform Group	MPN/100 ml
	Fecal Streptococci	MPN/100 ml
Miscellaneous Physical and Chemical Parameters	pH	pH units
	Temperature	°C
	Oil and Grease	mg/l
	Chlorides	mg/l
	Pesticides	mg/ml
	Surfactants	mg/l

a degree, the nature and size of these objects will be determined by the physical layout of the collection system. No estimate was made for the incidence of floating solids.

Suspended solids includes all those particles so fine or so bouyant that they are suspended in the stream. It may include both particles which will settle following loss of stream velocity and those which are either colloidal or at neutral bouyancy. These solids may or may not be volatile in nature. The volatile solids are generally composed of various organic substances, both animal and vegetable in origin, which are characterized by putrescibility and eventual decomposition into simpler, stable materials. Volatile solids of vegetable origin may include cut and fallen vegetation (leaves, grass, etc.), humus, and discarded food. Animals contribute offal and carcasses of various sizes and stages of decomposition. Nonvolatile suspended solids are primarily soils and general surface debris and are inherently stable and inorganic in nature. Analysis of runoff from similar watersheds in the Cincinatti study cited earlier led to the tentative assumption that, after construction is completed, runoff from Columbia will probably not exceed 300 mg/l suspended solids, 60 mg/l of which might be volatile.

Dissolved solids may include various soluble minerals and salts, as well as small quantities of other organic and inorganic materials which might be slightly soluble. Decomposition of volatile suspended solids often produces soluble organic products. Some dissolved materials have been precipitated from the atmosphere by the rainfall and were dissolved before striking the ground, but most have entered the water as suspended solids and have subsequently dissolved. Many dissolved substances are of interest themselves as specific pollutants, such as chlorides, pesticides, surfactants, etc. Since these separately discussed pollutants are the only dissolved solids expected to be of any interest in the Columbia runoff, no limit was created for dissolved solids.

(2) Oxygen Balance. Biochemical Oxygen Demand, or BOD, is an index determined by a laboratory procedure for measuring the amount of oxygen consumed, in five days, by natural agencies in stabilizing the volatile material in the water. It is a reasonable estimate of the oxygen that will be consumed in nature as the various organic materials decompose. Untreated domestic sewage has a BOD on the order of 200 mg/l, while Maryland Department of Water Resources water quality criteria classify as "good" a stream having less than 3.5 mg/l BOD. It was estimated that the storm runoff might contain up to 25.0 mg/l BOD.

Chemical Oxygen Demand (COD) measures the amount of oxygen consumed under specified conditions in the oxidation of organic and oxidizable inorganic matter, corrected for the influence of chlorides. COD and BOD tend to complement one another, although there is no inherent relation between the two. The difference is primarily one of the mechanism of oxidation. In the present analysis, the BOD test is considered more appropriate than COD, so no estimate was made of storm water COD.

Dissolved Oxygen (DO) measures the quantity of oxygen available in the water in molecular form. It may range from 0.0 up to saturation, which is a function of water temperature and, to a lesser extent, atmospheric pressure. Saturation levels may range, at sea level, from 7.1 mg/l at 35°C to 14.4 mg/l at 1°C. It can be assumed that falling raindrops are 100 percent saturated and that any loss of oxygen content is a result of rising temperature or of physical contact with oxidizable contaminants, or both. It was assumed that the DO of storm water might be as low as 0.0 mg/l.

(3) Nutrients. Both nitrogen and phosphorous are effective nutrients and are commonly found in water as nitrates and phosphates. Nitrogen might also be present as nitrites and as dissolved or organic nitrogen, due to the decomposition of organic material. The presence of either of these materials is usually indicative of decomposing organic materials in the stream or in the watershed, or of the direct application of chemical fertilizers containing these materials to land in the watershed, resulting in a high nutrient level of runoff. The presence of these substances in excessive concentrations contributes to algal blooms and to distortion of the ecological balance of the receiving body of water. It was assumed that the runoff under study would not exceed 3.0 mg/l. Nitrogen (measured as NO_3) or 1.0 mg/l phosphorous (measured as PO_4).

(4) Bacteria. Due to the great difficulty of isolating and identifying all of the possible pathogenic (disease-producing) bacteria which might be present in storm water, it is common practice to test for presence of one of the fecal bacteria and use that strain as an indicator of fecal contamination. If such contamination is found to be present, it is considered possible that pathogens might also be present. If no fecal bacteria are identified, the presence of pathogens is considered unlikely. The fecal bacteria used most widely as an indicator is the coliform group of bacteria, particularly the Escherichia Coli, a relatively large rod-like bacteria. It is readily detected by means of any of several standard laboratory methods. Since coliform bacteria may be present in soil as well as humans and animals, more specific indicators, such as the fecal streptococci group, are sometimes used. In all cases, test results are reported as most probable number (MPN) per 100 ml. Since public health considerations dictate that chlorination should be practiced regardless of the bacterial concentrations, no limit was suggested for the probable bacterial pollution.

(5) Miscellaneous Physical and Chemical Parameters. From the many parameters that could be considered under this heading, the only ones which are expected to be of any significance in

the Wilde Lake area are chlorides, oil, and grease. No quantitative estimate was made for oil and grease, but it is virtually certain that runoff from streets, driveways and parking lots will include noticeable quantities. It is estimated that chlorides, primarily from salt used to melt snow in the winter, may range as high as 160 mg/l. Table II-11 lists the tentative storm water quality description for the Wilde Lake watershed.

TABLE II-11. EXPECTED MAXIMUM POLLUTANT CONCENTRATIONS STORM WATER WILDE LAKE WATERSHED

Type	Pollutant		Expected Max. Concentration	
Solids	Floating Solids		Present	
	Suspended Solids		300	mg/l
	Volatile Suspended Solids		60	mg/l
Oxygen Balance	BOD		25	mg/l
Nutrients	Nitrogen (N, NO_2, NO_3)	(as NO_3)	3.0	mg/l
	Phosphorous (P, PO_4)^3	(as PO_4^3)	1.0	mg/l
Miscellaneous Physical and Chemical Parameters	Chlorides		160	mg/l
	Oil and Grease		Present	

2. Field Sampling Results

In order to permit early verification of the expected pollutant concentrations listed in Table II-11, a sampling point was established in the Wilde Lake watershed in conjunction with the gaging station described in Section II.C.9. An automatic sampler was used to collect runoff samples at one-hour intervals from a point immediately upstream from the weir pond. These samples were correlated with measured stream flow at the time of sampling and analyzed for various pollutants by Hittman Associates. The results of this analysis are presented as Tables II-12, II-13, and II-14. Tables II-12 and II-14 reflect water quality during two of the storms whose hydrographs were analyzed in Section II.C.9, identified as Storms 1 and 3. Table II-13 lists data on water quality during a typical period of minimum flow, when no storm runoff was present.

All samples were excessively high in suspended and total solids, due to a high level of construction activity in the watershed during the period under study. A large portion of the flood plain itself had been denuded as a result of the construction of a sanitary sewer and erosion was severe. For this reason, the results obtained cannot be treated as typical of this area. Furthermore, little urbanization has occurred on the gaged watershed at the present time. At the time of sampling, facilities were

TABLE II-12. WATER QUALITY VERSUS RUNOFF RATE (GAGING STATION) JUNE 26 TO JUNE 27, 1968 (STORM 1)

Sampling Time	Runoff Rate cfs	Turbidity JTU	Suspended Solids mg/1	Settable Solids mg/1	Total Solids mg/1	Chemical Oxygen Demand mg/1	pH	Alkalinity $(CaCO_3)$ mg/1	Hardness $(CaCO_3)$ mg/1	Chloride (Cl^-) mg/1	Nitrate (Nitrite) mg/1	Phosphate $(PO_4^=)$ mg/1
16:30	0.04	190	140	----	252	20	8.3	55	50	8	0.7	---
17:30	0.04	125	---	----	---	--	---	--	--	----	---	---
18:30	0.04	78	50	----	---	--	8.3	--	--	----	---	---
19:30	0.04	130	---	----	---	20	8.3	55	50	7.5	0.5	---
20:30	0.04	82	75	----	185	--	---	--	--	----	---	---
21:30	0.04	103	---	----	---	--	---	--	--	----	---	---
22:30	0.04	85	80	----	---	30	7.9	55	45	7.5	0.6	3.4
23:30	1.00	4,500	11,450	10,830	11,650	40	7.1	20	30	24	0.1	0.3
24:30	1.65	3,500	5,780	5,250	---	--	7.2	20	25	12	0.0	1.1
01:30	0.60	1,500	1,950	1,550	2,100	30	7.3	25	25	11.5	0.2	---
02:30	0.44	1,050	1,105	920	---	20	---	--	--	----	---	---
03:30	0.33	820	823	----	970	30	7.6	25	30	12	0.3	---
04:30	0.23	325	---	----	---	--	---	--	--	----	---	---
05:30	0.20	270	290	----	---	--	---	--	--	----	---	---
06:30	0.17	----	---	----	---	--	---	--	--	----	---	---
07:30	0.16	151	150	----	255	--	8.0	30	35	10	---	---
08:30	0.15	----	---	----	---	--	---	--	--	----	---	---
09:30	0.14	235	238	----	---	30	8.2	35	40	9.5	0.6	---
10:30	0.15	103	---	----	---	--	---	--	--	----	---	---
11:30	0.13	1,050	925	----	1,110	--	---	--	--	----	---	---
12:30	0.10	310	310	----	---	--	---	--	--	----	---	---
13:30	0.15	105	---	----	---	--	---	--	--	----	---	---
14:30	0.14	1,300	1,500	1,220	1,680	30	7.2	10	30	10	0.5	1.7

TABLE II-13. WATER QUALITY VERSUS RUNOFF RATE (GAGING STATION) JULY 12, 1968 (BASE FLOW)

Sampling Time	Runoff Rate cfs	Turbidity JTU	Suspended Solids mg/l	Settable Solids mg/l	Total Solids mg/l	Chemical Oxygen Demand mg/l	pH	Alkalinity ($CaCO_3$) mg/l	Hardness ($CaCO_3$) mg/l	Chloride (Cl^-) mg/l	Nitrate (Nitrite) mg/l	Phosphate ($PO_4^{=}$) mg/l
02:00	0.04	32	15	----	---	--	7.55	--	--	---	----	----
04:00	0.04	35	12	----	130	15	7.55	45	30	7.5	0.32	0.10
06:00	0.04	35	10	----	---	--	7.55	--	--	---	----	----
08:00	0.04	32	10	----	129	20	7.55	40	30	7.5	0.30	0.10
10:00	0.04	38	14	----	---	--	7.60	--	--	---	----	----
12:00	0.04	45	21	----	140	15	7.55	40	33	7.5	0.29	0.08
14:00	0.04	38	16	----	---	--	7.55	--	--	---	----	----
16:00	0.04	40	18	----	138	10	7.55	40	30	7.5	0.27	0.12
18:00	0.04	35	11	----	---	--	7.60	--	--	---	----	----
20:00	0.04	38	8	----	122	15	7.55	43	35	7.5	0.25	0.10
22:00	0.04	30	12	----	---	--	7.60	--	--	---	----	----
24:00	0.04	35	16	----	130	10	7.60	40	30	7.5	0.27	0.09

TABLE II-14. WATER QUALITY RUNOFF RATE (GAGING STATION) JULY 15, 1968 (STORM 3)

Sampling Time	Runoff Rate Q cfs	Turbidity JTU	Suspended Solids mg/l	Settable Solids mg/l	Total Solids mg/l	Chemical Oxygen Demand mg/l	pH	Alkalinity ($CaCO_3$) mg/l	Hardness ($CaCO_3$) mg/l	Chloride (Cl^-) mg/l	Nitrate (Nitrite) mg/l	Phosphate ($PO_4^{=}$) mg/l
13:00	0.05	30	10	------	128	10	7.50	40	30	7.5	0.28	0.10
14:30	0.05	35	20	------	-----	10	6.95	43	30	7.5	0.31	0.30
15:10	19.50	5,200	23,800	23,300	23,950	10	6.55	15	30	15.0	0.15	0.07
15:20	12:50	4,600	14,600	13,600	-----	30	6.35	15	25	12.5	0.34	1.90
15:30	8.20	4,300	10,500	9,800	10,680	30	6.25	10	20	22.5	0.60	4.60
15:45	4.70	4,000	7,750	7,100	-----	20	6.50	8	25	10	0.20	0.50
16:00	2.70	3,600	6,000	5,380	6,200	40	6.80	10	25	22.5	0.60	5.30
16:15	2.20	2,600	3,900	3,500	-----	20	7.00	11	20	12.5	0.35	2.20
16:30	1.00	2,300	3,040	------	3,170	30	7.10	10	25	10	0.34	0.15
17:30	0.30	1,100	955	------	-----	40	7.00	10	23	9	0.45	0.18
18:30	0.30	1,150	1,030	------	-----	20	7.05	18	25	9	0.27	0.27
19:30	0.20	750	725	------	-----	10	7.15	20	28	9	0.28	0.20
20:30	0.20	1,900	1,510	------	1,680	10	7.25	25	35	14	0.32	0.15
14:30	0.08	58	50	------	-----	5	7.45	35	30	7.5	0.28	0.25

not available for determining BOD and a COD test was employed instead. Due to the atypical conditions existing, however, the results of the tests during storm flows are judged to have little value with respect to estimating future pollution loads. The data presented of Table II-13 is of some interest in that the base flow water quality might be expected to be relatively independent of watershed conditions. As the data indicates, this flow was found to be of excellent quality, well within the expected limits. The continuation of this sampling activity throughout the period of urbanization of this watershed would be of considerable value in assessing the effect of construction and subsequent population on runoff quality.

3. Water Quality Standards for Wilde Lake

The Maryland Department of Water Resources has recently adopted a comprehensive set of water quality standards regulating the state's interstate and principal intrastate waters, in compliance with Federal requirements (Ref. 40). Although Wilde Lake is not specifically assigned a minimum water quality in these standards, its intended use is clearly consistent with the Class C uses outlined in the regulations--water contact recreation, fish propagation, agricultural and industrial water supply. These standards make the following requirements of bodies of water intended for Class C uses:

(a) Fecal Coliform Bacteria less than 240 MPN/100 ml

(b) Dissolved Oxygen greater than or equal to 4.0 mg/l at any time, 5.0 mg/l monthly average

(c) pH greater than or equal to 6.0, less than or equal to 8.5

(d) Temperature less than or equal to 93°F (35°C)

In addition, general requirements are made, prohibiting harmful concentrations of any other pollutants which might be present. Harmful concentrations are to be determined in terms of the intended uses of the body of water which are to be protected.

A conservative interpretation of these water quality standards and the ability of Wilde Lake to neutralize pollutants entering it from storm water runoff led to the definition of effluent standards for all water deliberately discharged into Wilde Lake or its tributary watercourses. These standards take into account the possibility of spills during severe storms and have been further adjusted to coincide with the water quality criteria adopted for Class C water uses in the previous section. Table II-15 lists the parameters being used to define effluent water quality together with the maximum concentration of each.

TABLE II-15. EFFLUENT STANDARDS FOR WATER DISCHARGED INTO WILDE LAKE*

Type	Pollutant	Maximum Concentration
Solids	Floating Solids	None
	Suspended Solids	30.0 mg/l
	Volatile Suspended Solids	None
Oxygen Balance	BOD	None
Nutrients	Nitrogen (as NO_3)	50.0 mg/l
	Phosphorous (as PO_4)	1.0 mg/l
Bacteria	Fecal Coliform	240 MPN/100 ml
Miscellaneous Physical and Chemical Parameters	pH	6.0 or greater
	Oil and Grease	None
	Chlorides	250 mg/l
	Surfactants (ABS)	0.5 mg/l
	Ammonia (NH_3)	0.5 mg/l
	Arsenic	0.05 mg/l
	Barium	1.0 mg/l
	Cadmium	0.01 mg/l
	Calcium	200 mg/l
	Carbon Chloroform Extract	0.2 mg/l
	Chromium (hexavalent)	0.05 mg/l
	Copper	1.5 mg/l
	Cyanide	0.2 mg/l
	Flouride	3.0 mg/l
	Iron	0.3 mg/l
	Lead	0.1 mg/l
	Magnesium	150 mg/l
	Manganese	0.5 mg/l
	Phenolic Compounds (as phenols)	0.002mg/l
	Selenium	0.05 mg/l
	Silver	0.05 mg/l
	Sulfate	400 mg/l
	Zinc	15 mg/l
	Color	30 color units

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*Refs. 40 and 41

F. Storm Water Storage

1. Introduction

The Wilde Lake watershed of Columbia is similar to other recent suburban developments in its approach to the storm drainage problem. The natural drainage channels have been retained with their original topography and the flood plains are used, in part, to satisfy the open space requirement of the community planning concept. The flood plains are normally surrounded with building lots and these, in turn, are circumscribed by roads. The storm drain system consists of many, relatively small inlet and pipe systems, each draining independently into a flood plain. After discharge, the storm water follows natural drainage lines to Wilde Lake. Roads are generally located at or near the ridge lines of the various sub-drainage areas and only a few cases exist within the Wilde Lake watershed where an appreciable amount of runoff is collected by a single closed collection system before discharge to a flood plain.

2. Construction Types

The early planning of the Local Reuse System study provided for consideration of unconventional storage facilities, such as natural or artificial underground cavities, slip-formed vaults beneath streets, etc., as well as more conventional approaches to storage within the flood plain. Examination of the drainage area indicated that significant quantities of storm water were not available at any place in the watershed, except at locations within the natural flood plains. Of the 22 sub-watersheds considered, only number 10 enters Wilde Lake as a wholely closed system without having been substantially collected within a natural flood plain. When storage is to be provided in the flood plain, two types were considered - open ponds and constructed storage. In sub-watersheds where flood plain space is not available, such as number 10, conventional constructed storage was considered, along with unconventional approaches, such as special vaults under roads and parking lots. Construction costs for such vaults were found to be on the order of $0.20/gallon of storage capacity and maintenance problems were considered to be severe. Since conventional constructed storage is competitive with other methods from sizes of 50,000 gallons upward and maintenance is facilitated by the improved access, parametric costs for unconventional approaches were not developed and no further consideration of them was judged necessary.

In the case of constructed storage, costs are dependent on location with respect to existing grade. In most cases, the storage facility must be placed below grade to permit gravity inflow under all conditions. Occasionally, local topography may permit a tank to be constructed partially or completely above grade. To maintain maximum flexibility, the below grade and above grade cases were separately analyzed and costed. Tanks of steel and of prestressed concrete construction were also investigated. Open ponds were studied and the optimum combination of excavated and natural storage determined for ponds in topography similar to Columbia. Table II-16 summarizes the types of reservoir construction which were studied and cost estimated.

TABLE II-16

STORM WATER STORAGE RESERVOIR CONSTRUCTION TYPES

Natural Storage	Open Ponds
Constructed Storage	Steel Tanks - Above Grade Steel Tanks - Below Grade Concrete Tanks - Above Grade Concrete Tanks - Below Grade

3. Sizing Criteria

The storm water storage facilities included in the Local Storage, Treatment, and Reuse of Storm Water concept function as multiple purpose reservoirs and as such, are subject to several potentially conflicting requirements. Their basic purposes in the order of the priority implied by the orientation of this study, are water quality control, water supply, and the creation of a pleasing and harmonious asset to the local environment. As water quality control reservoirs, they are expected to be designed and operated so as to prevent degradation of their contents, maintain sufficient release to the natural watercourse to protect downstream uses, and possibly effect sedimentation. The water supply use dictates certain relationships of storage capacity, supply, and yield, dependent on, among other things, the reliability with which the yield must be maintained. Finally, the appearance of the facility must be at least consistent with its surroundings and might well be designed to be an asset to the area. The presence of the facilities must be acceptable to the community and pose no particular safety hazard or nuisance.

In studying size requirements for the local storage units, the individual reservoir locations were first analyzed to determine the yield versus capacity relationships, assuming 100 percent reliability over the five year period of the synthetic hydrograph. Similar analyses were performed to determine relationships with 99, 98, 95, 90, and 80 percent reliability for the five year period. These analyses were performed for constant yields and for yields approximating the patterns of several different levels of lawn sprinkling use. Next, water quality considerations were analyzed in terms of operating and design requirements, and the storage units studied as sedimentation basins. Finally, sizes and design criteria were developed which met requirements imposed by all three purposes. The following sections describe in detail, the development of these criteria.

a. Water Supply Use

(1) Storage/Yield Characteristics. A common approach to water supply reservoir sizing problems is the Mass Diagram or Rippl Method (Ref. 42). When performed graphically, this method plots cumulative supply against cumulative yield so that cumulative surplus or deficiency can be examined. A simple graphical manipulation is sufficient to measure maximum required storage directly for the time period plotted. The identical result may be obtained with somewhat better resolution by applying the following continuity equations to tabular data:

$$Q_i + S_i = D_i + S_{i+1} \tag{II-25}$$

where:

Q_i = inflow on the i^{th} day

S_i = storage volume at the start of the i^{th} day

D_i = yield on the i^{th} day

This equation may be applied to each successive time period throughout the available record and the maximum value found for S_i is the maximum required storage, comparable to that found by the graphical technique. In order to achieve meaningful results in small watersheds subject to large, short-term fluctuations in runoff rates, a five year daily synthetic hydrograph was developed (Section II. C. 8), containing 1825 data points. This data was developed for each of 22 sub-watersheds. Each of these must be analyzed over a wide range of demand levels, or yields. This last requirement is made to permit an economic tradeoff between the revenue of additional water uses and the incremental cost of additional storage. A preliminary appraisal of the difficulty of a conventional mass balance continuity analysis of 1825 data points for each of the watershed yield combinations led to investigation of alternate approaches to the reservoir sizing problem.

The selected method was that of Residual Mass Tabulation, a technique for approximating the result of the method described above with many fewer calculations and in a manner much better suited to the high speed processing characteristics of the digital computer (Ref. 42). Table II-17 is a portion of a residual mass tabulation for one of the sub-watersheds in the Wilde Lake area. The two left-hand columns list total accumulated runoff (in cfs-days) for various periods (in days) in the synthetic record. The first line indicates runoff for the driest day of record, the second line indicates the total runoff for the driest five consecutive days of record, the third line indicates the total for the driest 10 consecutive days of record, etc. For example, the eleventh line indicates that during the driest 60 consecutive days of record, the total accumulated runoff was 1.0 cfs-day. This listing continues until the period listed equals the total record, 1825 days, and the accumulated runoff equals the total runoff for the period, 83.09 cfs-day in this example. The second pair of columns, headed "DEMAND (CFS)= 0.0125", show the method of analysis for a constant yield of 0.0125 cfs. The column headed "OUTFLOW" lists the total yield for each of the periods listed in the first column (number of days times daily yield). The column headed "STORAGE" lists the volume drawn from storage found by subtracting the "RUNOFF" column from the "OUTFLOW" column. The largest positive number in this column is the approximate size, in cfs-day, of the required reservoir, given this demand. This process is repeated successively with various demands until the entire possible operating range has been explored. In each case, the largest positive number in the storage column is taken as the required reservoir size. When the "STORAGE" column does not show a negative entry in the last row, a supply deficit is indicated. That is, the total available supply is less then the total required yield and no finite reservoir can be constructed which would provide this yield during the period analyzed.

TABLE II-17. EXAMPLE OF RESIDUAL MASS TABULATION - SUB-WATERSHED #8

Period (days)	Runoff (cfs-days)	Demand(cfs)=0.0125 Outflow (cfs-days)	Demand(cfs)=0.0125 Storage (cfs-days)	Demand(cfs)=0.0200 Outflow (cfs-days)	Demand(cfs)=0.0200 Storage (cfs-days)	Demand(cfs)=0.0400 Outflow (cfs-days)	Demand(cfs)=0.0400 Storage (cfs-days)	Demand(cfs)=0.0600 Outflow (cfs-days)	Demand(cfs)=0.0600 Storage (cfs-days)
1	0.0	0.0125	0.0125	0.0200	0.0200	0.0400	0.0400	0.0600	0.0600
5	0.0	0.0625	0.0625	0.1000	0.1000	0.2000	0.2000	0.3000	0.3000
10	0.0	0.1250	0.1250	0.2000	0.2000	0.4000	0.4000	0.6000	0.6000
15	0.0	0.1875	0.1875	0.3000	0.3000	0.6000	0.6000	0.9000	0.9000
20	0.0	0.2500	0.2500	0.4000	0.4000	0.8000	0.8000	1.2000	1.2000
25	0.0	0.3125	0.3125	0.5000	0.5000	1.0000	1.0000	1.5000	1.5000
30	0.0	0.3750	0.3750	0.6000	0.6000	1.2000	1.2000	1.8000	1.8000
35	0.12	0.4375	0.3175	0.7000	0.5800	1.4000	1.2800	2.1000	1.9800
40	0.22	0.5000	0.2800	0.8000	0.5800	1.6000	1.3800	2.4000	2.1800
50	0.37	0.6250	0.2550	1.0000	0.6300	2.0000	1.6300	3.0000	2.6300
60	1.00	0.7500	-0.2500	1.2000	0.2000	2.4000	1.4000	3.6000	2.6000
80	1.54	1.0000	-0.5400	1.6000	0.0600	3.2000	1.6600	4.8000	3.2600
100	1.70	1.2500	-0.4500	2.0000	0.3000	4.0000	2.3000	6.0000	4.3000
150	3.32	1.8750	-1.4450	3.0000	-0.3200	6.0000	2.6800	9.0000	5.6800
200	4.65	2.5000	-2.1500	4.0000	-0.6500	8.0000	3.3500	12.0000	7.3500
250	7.53	3.1250	-4.4050	5.0000	-2.5300	10.0000	2.4700	15.0000	7.4700
300	9.97	3.7500	-6.2200	6.0000	-3.9700	12.0000	2.0300	18.0000	8.0300
350	12.33	4.3750	-7.9550	7.0000	-5.3300	14.0000	1.6700	21.0000	8.6700
400	13.85	5.0000	-8.8500	8.0000	-5.8500	16.0000	2.1500	24.0000	10.1500
450	15.58	5.6250	-9.9550	9.0000	-6.5800	18.0000	2.4200	27.0000	11.4200
500	10.00	6.2500	-11.7500	10.0000	-8.0000	20.0000	2.0000	30.0000	12.0000
600	23.05	7.5000	-15.5500	12.0000	-11.0500	24.0000	0.9500	36.0000	12.9500
700	29.52	8.7500	-20.7700	14.0000	-15.5200	28.0000	-1.5200	42.0000	12.4800
800	33.71	10.0000	-23.7100	16.0000	-17.7100	32.0000	-1.7100	48.0000	14.2900
900	39.03	11.2500	-27.7800	18.0000	-21.0300	36.0000	-3.0300	54.0000	14.9700
1000	43.37	12.5000	-30.8700	20.0000	-23.3700	40.0000	-3.3700	60.0000	16.6300
1200	52.26	15.0000	-37.2600	24.0000	-28.2600	48.0000	-4.2600	72.0000	19.7400
1400	61.56	17.5000	-44.0600	28.0000	-33.5600	56.0000	-5.5600	84.0000	22.4400
1600	72.27	20.0000	-52.2700	32.0000	-40.2700	64.0000	-8.2700	96.0000	23.7300
1825	83.09	22.8120	-60.2780	36.5000	-46.5900	73.0000	-10.0900	109.5000	26.4100
Maximum Storage			0.3750		0.6300		3.3500	(supply deficit)	

After the residual mass tabulation has been performed for a sufficient number of yields to describe the range of possible operation, a plot is made of yield versus storage requirement. A typical plot is shown on Figure II-30. In this case, the units of storage have been converted from cfs-day to gallons and those of yield from cfs to gallons/day. Since the method used samples various time periods and computes storage for the sampled periods only, the storage requirement found will be only an approximation of the actual requirement. It may be equal to the actual requirement or more probably, slightly smaller. For this reason, the plot shows a smooth curve intersecting or passing above the plotted points. This curve can be taken as a continuous relationship between storage requirements and yield for the period of the synthetic hydrograph used to generate the points. Figure II-31 shows the storage/yield characteristic of another sub-watershed as well as the results of a graphical Rippl Method solution for the same area. The correspondence between the two methods was considered satisfactory for the purposes of this study and all subsequent storage analyses were made by the method of residual mass tabulation, using digital computers.

(2) Storage/Yield Characteristics for Sprinkling Uses. The determinations of storage requirements described above assumed that the demand on the storage facility or yield, would be constant in terms of volume/day. This assumption is relatively valid for domestic uses such as toilet flushing, bathing, laundering, food preparation, etc. The only significant use which cannot be treated as a constant demand is lawn sprinkling. In addition to its totally seasonal characteristic in many areas of the country, the pattern of use is highly irregular from day to day. In their major study on residential water use, Linaweaver and others found the following relationships between the most severe and the average periods of sprinkling use in a residential area near the Columbia site (Ref. 27):

TABLE II-18. RATIO OF SPRINKLING USE DURING SELECTED PERIOD TO AVERAGE SPRINKLING USE

Glenmont and Northwest Branch Estates,
Washington Suburban Sanitary Commission,
October 1963 to September 1965

Time Period	Ratio of Sprinkling Use to Average Use (Ref. 27, p. 6)
Peak Hour	21.20
Maximum Day	7.17
Maximum 92 Days	2.16
365 Days	1.00

A continuous curve was fitted to the four data points listed by a parabolic regression technique, yielding the following equation:

$$R_i = 7.26\, d^{-0.336*} \qquad \text{(II-26)}$$

*Standard error of estimate = 0.0061; $\rho = 0.993$

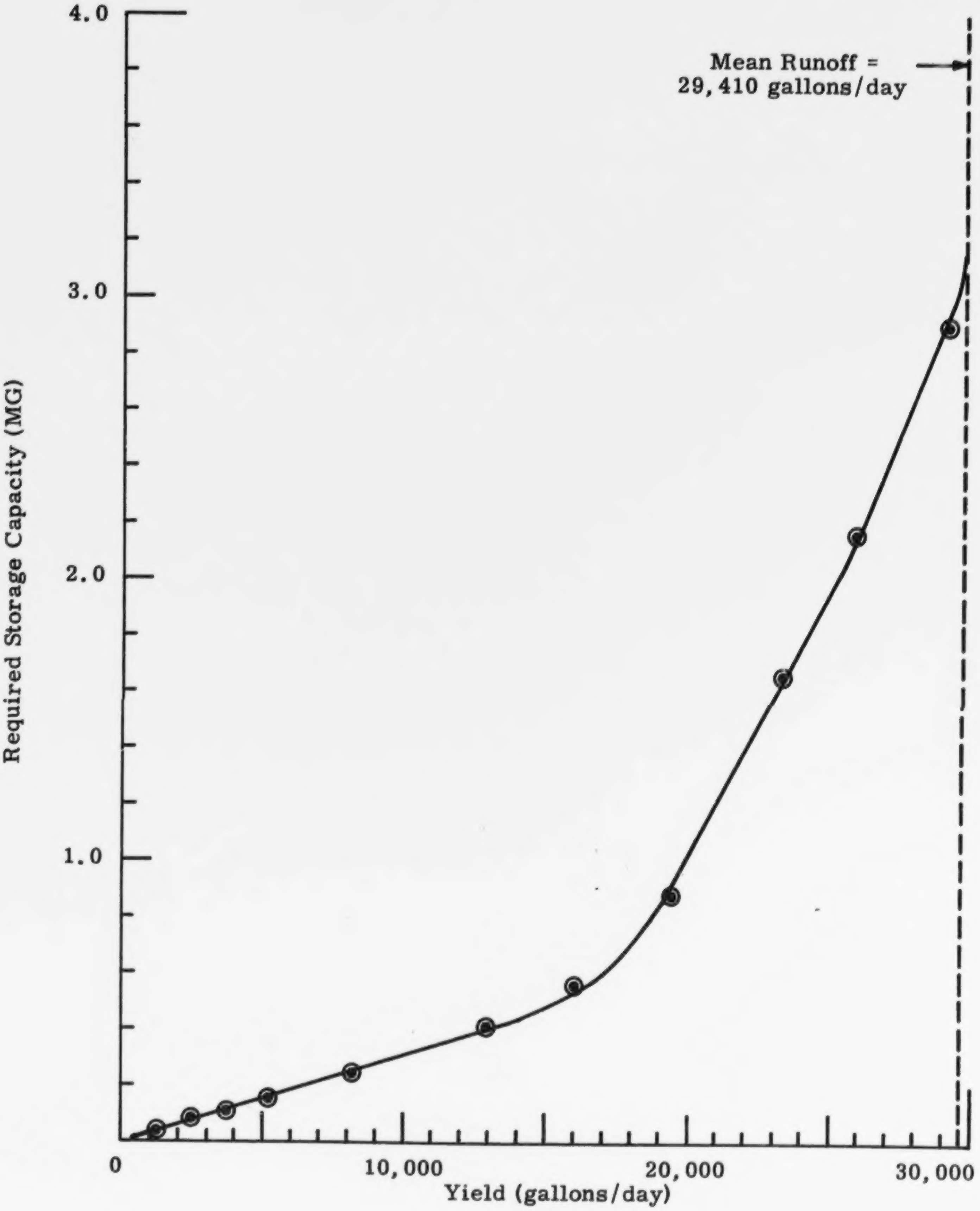

Figure II-30. Storage vs. Yield
Example of Residual Mass Tabulation Analysis
Sub-Watershed Number 8

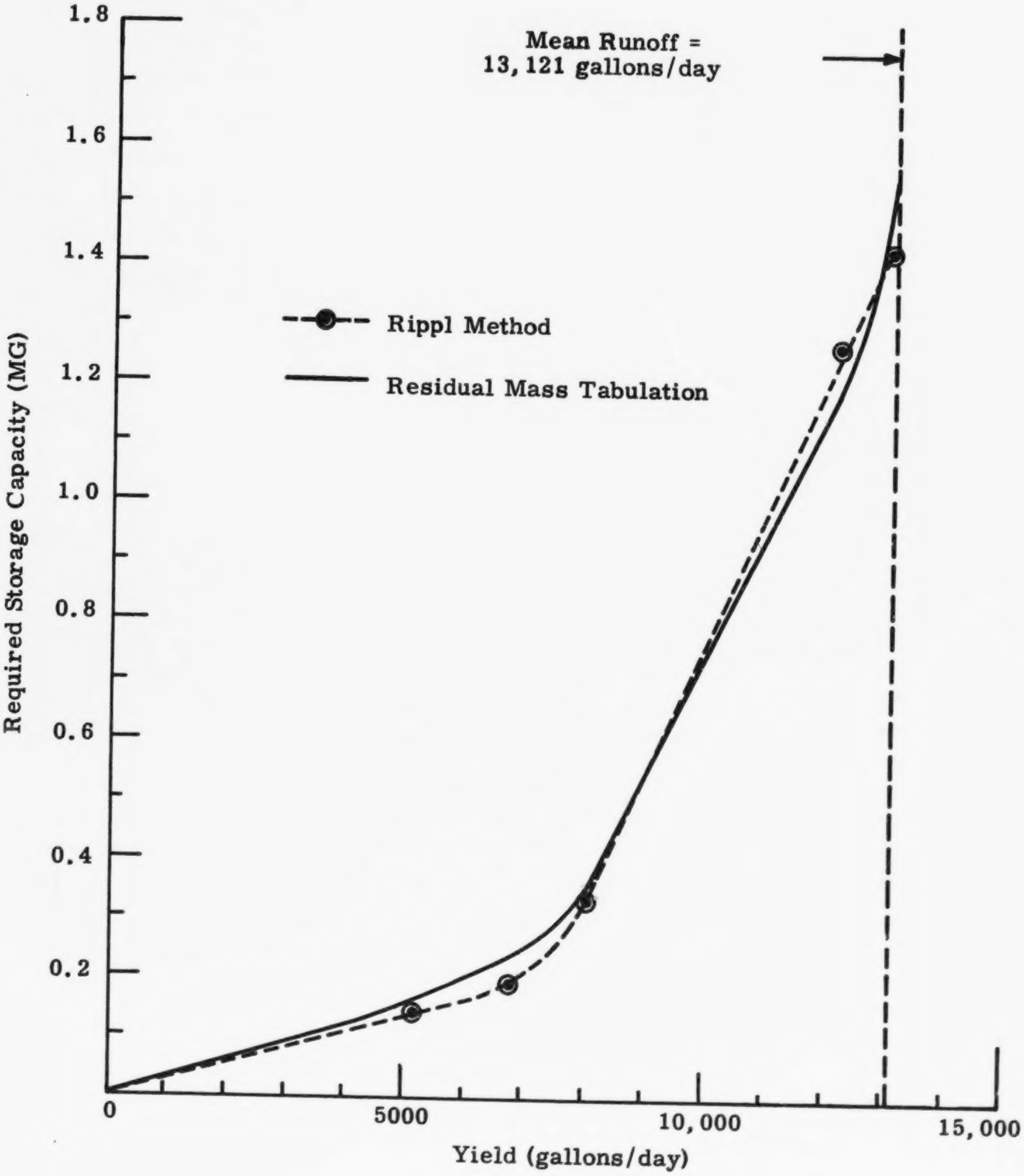

Figure II-31. Storage vs. Yield - Comparison of Rippl Method and Residual Mass Tabulation, Sub-Watershed Number 4

where:

R = ratio between average sprinkling use during period, d, and annual average sprinkling use

d = period of sprinkling use being studied in days

This equation can now be used to estimate the ratio of sprinkling use during the most severe period of a selected length to the average sprinkling use. When average sprinkling use levels are developed in Section II.D and applied to this expression, the most severe sprinkling use can be evaluated for any period. Since the average use levels and ratios employed have been selected from studies of similar residential areas in the same geographical and climatological area, only the irrigable area is required as an input to this calculation.

This approach to evaluation of sprinkling demand patterns can be applied to the residual mass tabulation method of reservoir design to determine the reservoir requirements for a demand that is to be entirely or partially dedicated to sprinkling uses. The only additional assumption required is that the most severe sprinkling demands will coincide with the driest periods of record. Although dry periods may occur during seasons when evapotranspiration is very low and no sprinkling would be practiced, the assumption is clearly a conservative one and can be observed to conform to experience in a high percentage of actual cases. Table II-19 shows the modified tabulation for a sub-watershed with parallel analysis of a 15,000 gallon/day average yield directed to uses which are zero percent sprinkling, 25 percent sprinkling, and 100 percent sprinkling. The "RATIO" columns are determined by evaluating Equation II-26 for each time period and adjusting this ratio as follows:

$$R_{i'} = p R_i + (1-p) \qquad \text{(II-27)}$$

where:

$R_{i'}$ = adjusted ratio for i^{th} period

R_i = ratio from Equation II-26 for i^{th} period

p = fraction of use assumed as sprinkling

These adjusted ratios are then entered in the "RATIO" columns in the rows corresponding to the correct time period. The tabulation proceeds as before, except that the "DEMAND" Column is the product of the average daily demand, the number of days in the period, and the corresponding adjusted ratio. The results of this tabulation indicate a significant increase in required storage as a direct result of an increasing proportion of the water being used to irrigate lawns. Figure II-32 presents the results of a complete tabulation for one sub-watershed showing a family of curves for various proportions of sprinkling use. Figure II-33 is another presentation of the same data, comparing sprinkling use fraction to required storage for a range of total average daily use levels. Analyses of this type were used to derive the derating factor for sprinkling uses discussed in Section II.D.

TABLE II-19. EXAMPLE OF MODIFIED RESIDUAL MASS TABULATION - STORAGE VS. PROPORTION OF DEMAND USED FOR SPRINKLING LAWNS

Sub-watershed #16, Demand (cfs) = 0.0232
(g/d) = 15,000

Period	Runoff	0% Sprinkling			25% Sprinkling			100% Sprinkling	
(days)	(cfs-days)	Outflow	Storage	Ratio	Outflow	Storage	Ratio	Outflow	Storage
1	0.0	0.0232	0.0232	2.565	0.0595	0.0595	7.260	0.1684	0.1684
5	0.0	0.1160	0.1160	1.807	0.2096	0.2096	4.229	0.4906	0.4906
10	0.0	0.2320	0.2320	1.588	0.3684	0.3684	3.351	0.7774	0.7774
15	0.0	0.3480	0.3480	1.481	0.5154	0.5154	2.924	1.0176	1.0176
20	0.0	0.4640	0.4640	1.414	0.6561	0.6561	2.654	1.2315	1.2315
25	0.0	0.5800	0.5800	1.366	0.7923	0.7923	2.463	1.4285	1.4285
30	0.0	0.6960	0.6960	1.329	0.9250	0.9250	2.316	1.6119	1.6119
35	0.10	0.8120	0.7120	1.300	1.0556	0.9556	2.200	1.7864	1.6864
40	0.18	0.9280	0.7480	1.276	1.1841	1.0041	2.103	1.9516	1.7716
50	0.30	1.1600	0.8600	1.238	1.4361	1.1361	1.951	2.2632	1.9632
60	0.86	1.3920	0.5320	1.209	1.6829	0.8229	1.835	2.5543	1.6943
80	1.30	1.8560	0.5560	1.167	2.1660	0.8660	1.666	3.0921	1.7921
100	1.46	2.3200	0.8600	1.137	2.6378	1.1778	1.546	3.5867	2.1267
150	2.89	3.4800	0.5900	1.087	3.7828	0.8928	1.349	4.6945	1.8045
200	4.05	4.6400	0.5900	1.056	4.8998	0.8498	1.224	5.6794	1.6294
250	6.70	5.8000	-0.9000	1.034	5.9972	-0.7028	1.136	6.5888	-0.1112
300	9.12	6.9600	-2.1600	1.017	7.0783	-2.0417	1.069	7.4402	-1.6798
350	11.45	8.1200	-3.3300	1.004	8.1525	-3.2975	1.015	8.2418	-3.2082
Maximum Storage			0.8600			1.1778			2.1267

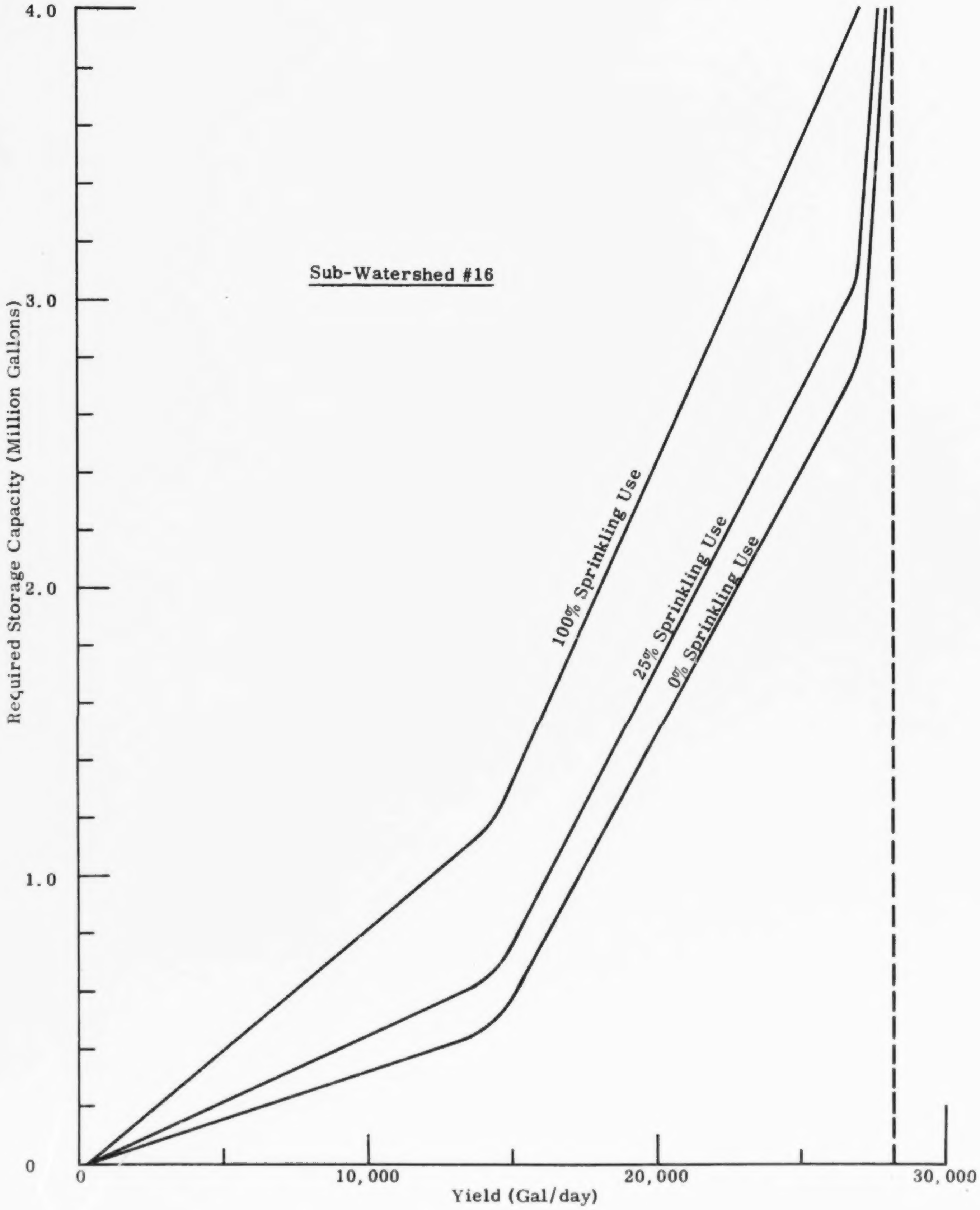

Figure II-32. Storage Yield Characteristic vs. Proportion of Demand Used for Sprinkling Lawns

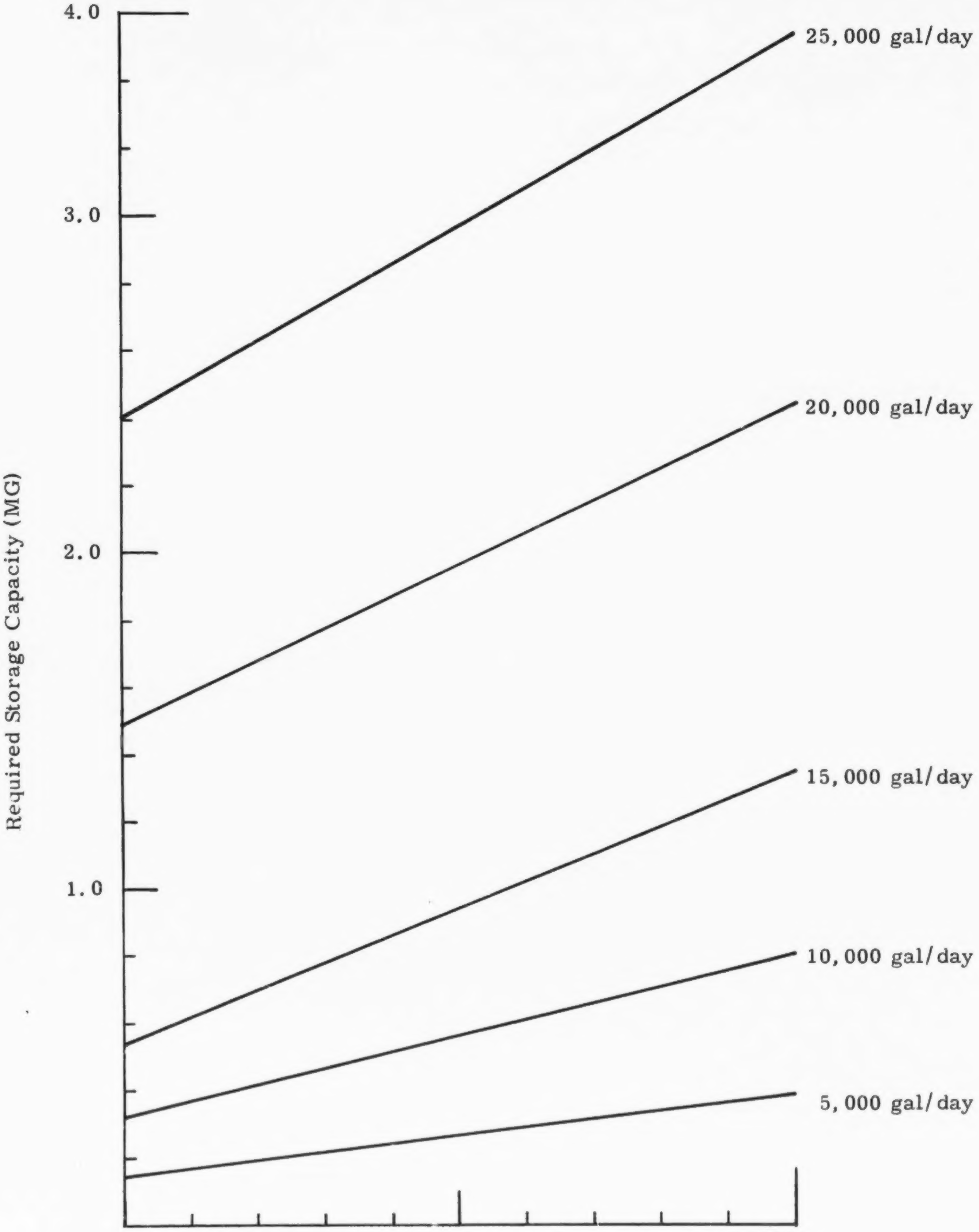

Figure II-33. Storage/Sprinkling Use Characteristic vs. Yield Sub-watershed Number 16

(3) Storage/Yield Characteristics as a Function of Reliability. Both the constant use and the sprinkling use storage characteristics investigated above have been oriented toward a requirement of 100 percent reliable operation of the storage system based on a five year synthetic hydrograph. Since the Local Storage, Treatment, and Reuse of Storm Water concept required that the system function only as a supplemental supply to a conventional public water distribution system, the storage requirements associated with various levels of system reliability were also analyzed. For purposes of this analysis, 90 percent reliability is defined as the ability of the system to supply at least 90 percent of the total water demand in any given year. Once again, the residual mass tabulation was employed to determine storage/yield characteristics. In this case, an assumption was made that the portion of the total demand which the storage system would fail to supply would be that occurring during the driest period. Table II-20 is an example of a tabulation for a sub-watershed with a 15,000 gallon/day total demand which would be either 100 percent, 98 percent, 95 percent, or 90 percent supplied by the reservoir under study. In each case, the demand column is reduced by the annual difference between reservoir yield and demand for each year or fraction thereof. In the example given, the 98 percent column is reduced by 0.1694 cfs-day* for all time periods of one year or less, by 2 x 0.1694 cfs-day for time periods of two years or less but more than one year, etc. A complete analysis of the sub-watershed for various levels of reliability is shown in Figure II-34. The same information is displayed in Figure II-35 as the required storage as a function of supply reliability for various levels of demand.

The previous paragraphs have summarized an investigation of storage requirements for the 22 sub-watersheds in the Wilde Lake area, derived as a continuous function of such variables as total demand, proportion of demand devoted to lawn sprinkling, and supply reliability. This analysis has indicated that storage requirements are highly nonlinear with demand, relatively sensitive to sprinkling uses, and highly sensitive to the availability of supplemental water from other sources. The method of analysis employed, a modified residual mass tabulation technique, was well suited to the application of digital computer methods and produced satisfactory results within the scope of the initial assumptions. Since the same simulated rainfall record was applied to all sub-watersheds, little error will be introduced by summing the cumulative runoff columns of the tabulations for several sub-watersheds to analyze the characteristics of the combination. Unfortunately, it is necessary to perform the complete tabulation for each possible combination of sub-watersheds in order to determine the storage/yield characteristic for each possible reservoir application.

b. Water Quality Criteria

(1) General Criteria. In providing for detention of water for any purpose, reasonable care must be taken to prevent degradation of the water in storage. In the application under study, possibilities exist for degradation as a result of decomposition of suspended or floating organic materials, flushing of previously settled solids into downstream flows, growth of algae in the case of open storage facilities, and the creation of

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*0.1694 cfs-day = $\left[(100 - 98)15{,}000/646{,}316 \times 100\right] 365$

TABLE II-20. EXAMPLE OF MODIFIED RESIDUAL MASS TABULATION - STORAGE VS. RELIABILITY

Sub-watershed #16, Demand (cfs) = 0.0232
(g/d) = 15,000

Period (days)	Runoff (cfs-days)	100% Reliability		98% Reliability		95% Reliability		90% Reliability	
		Outflow	Storage	Outflow	Storage	Outflow	Storage	Outflow	Storage
1	0.0	0.0232	0.0232	0.0	----	0.0	----	0.0	----
5	0.0	0.1160	0.1160	0.0	----	0.0	----	0.0	----
10	0.0	0.2320	0.2320	0.0626	0.0626	0.0	----	0.0	----
15	0.0	0.3480	0.3480	0.1786	0.1786	0.0	----	0.0	----
20	0.0	0.4640	0.4640	0.2946	0.2946	0.0405	0.0405	0.0	----
25	0.0	0.5800	0.5800	0.4106	0.4106	0.1565	0.1565	0.0	----
30	0.0	0.6960	0.6960	0.5266	0.5266	0.2725	0.2725	0.0	----
35	0.10	0.8120	0.7120	0.6426	0.5426	0.3885	0.2885	0.0	----
40	0.18	0.9280	0.7480	0.7586	0.5786	0.5045	0.3245	0.0810	-0.0990
50	0.30	1.1600	0.8600	0.9906	0.6906	0.7365	0.4365	0.3130	0.0130
60	0.86	1.3920	0.5320	1.2226	0.3626	0.9685	0.1085	0.5450	-0.3150
80	1.30	1.8560	0.5560	1.6866	0.3866	1.4325	0.1325	1.0090	-0.2910
100	1.46	2.3200	0.8600	2.1506	0.6906	1.8965	0.4365	1.4730	0.0130
150	2.89	3.4800	0.5900	3.3106	0.4206	3.0565	0.1665	2.6330	-0.2370
200	4.05	4.6400	0.5900	4.4706	0.4206	4.2165	0.1665	3.7930	-0.2570
250	6.70	5.8000	-0.9000	5.6306	-1.0694	5.3765	-1.3235	4.9530	-1.7470
300	9.12	6.9600	-2.1600	6.7906	-2.3294	6.5365	-2.5835	6.1130	-3.0070
350	11.45	8.1200	-3.3300	7.9506	-3.4994	7.6965	-3.7535	7.2730	-4.1770
Maximum Storage			0.8600		0.6906		0.4365		0.0130

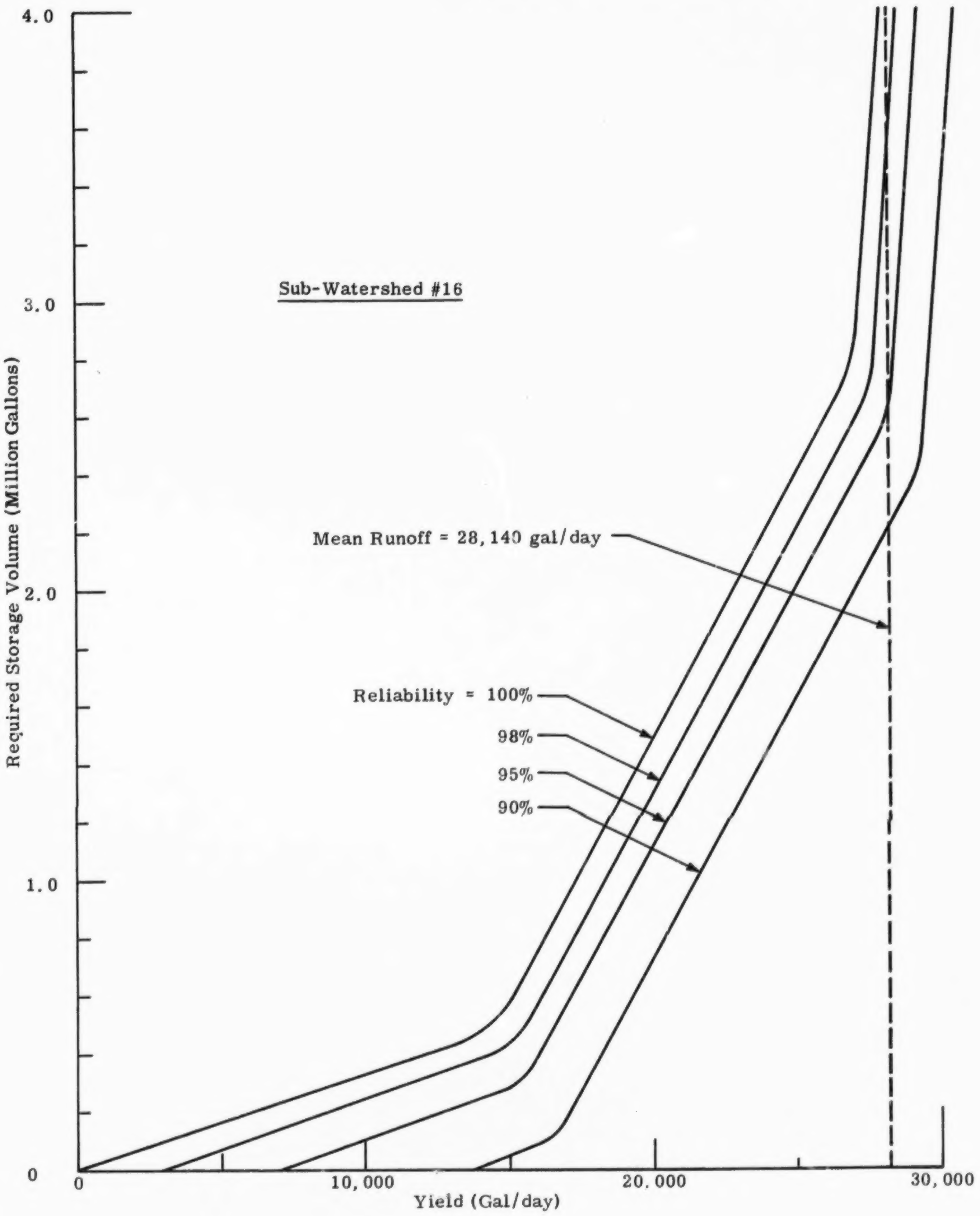

Figure II-34. Storage/Yield Characteristic vs. Reliability

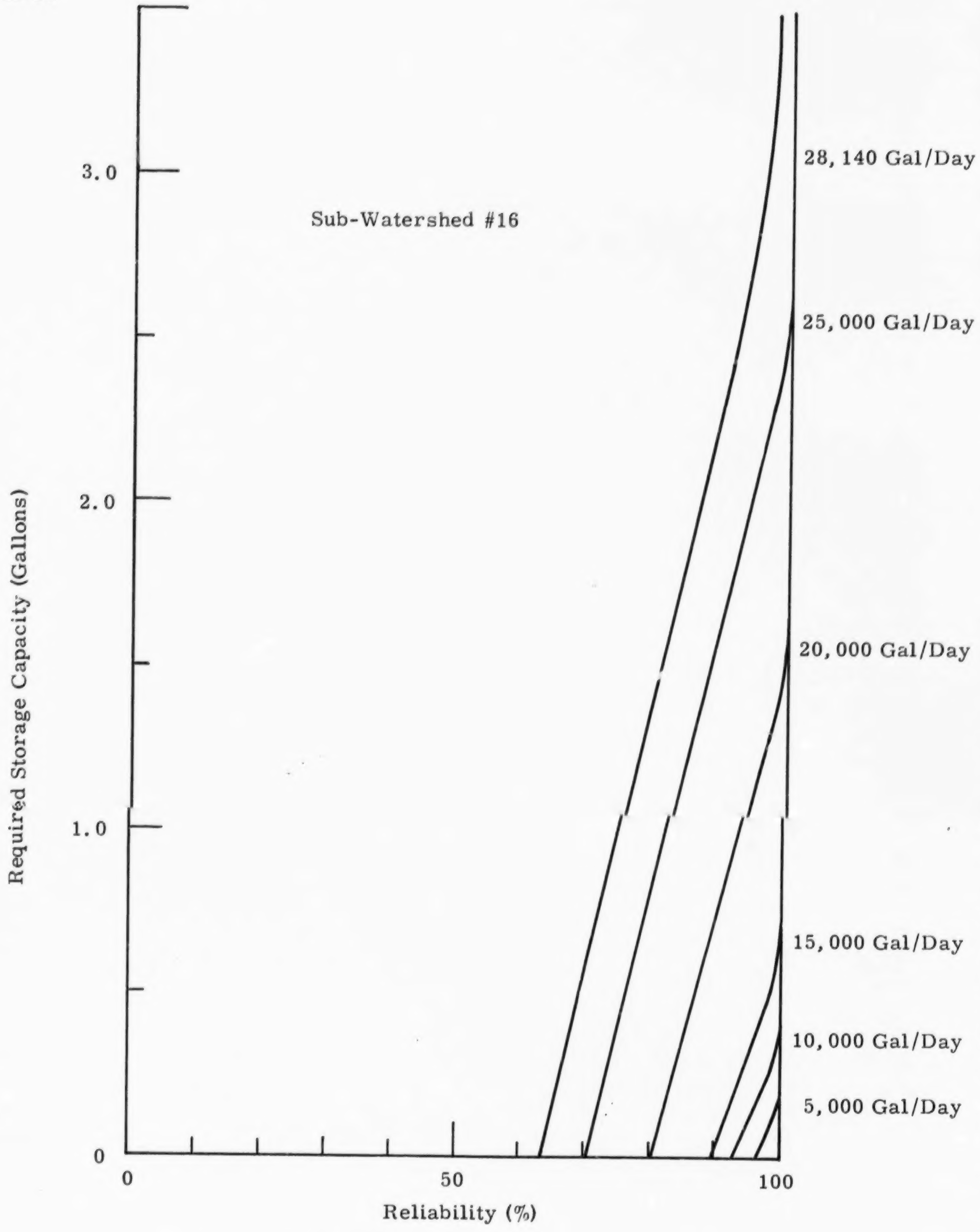

Figure II-35. Storage/Reliability Relationship vs. Average Daily Demand

nuisance conditions due to breeding of flies and mosquitoes from aquatic larvae. Fortunately, most organic materials found in storm water are not readily putrescible and may be retained for a reasonable period before appreciable decomposition occurs. Periodic removal of accumulated materials is normally sufficient to avoid difficulty from this source, as well as minimizing the possibility of flushing settled material downstream. Design criteria can be developed to minimize spills, further reducing the occurrence of this problem. Algal blooms can be controlled by chemical treatment and by the maintenance of a relatively high throughput as a proportion of average contents, which will serve to minimize the growth of insect larvae as well. Provision must be made for sufficient attention to the condition of the reservoir, such as daily inspections, to preclude the formation of a quality degradation condition.

It would be desirable to exclude trash from the storage facility by the installation of a mechanical trash screen at the storm water inlet. Such an installation, however, would be prone to accumulate trash and clog during periods of high flow, thus causing overflows and possible flood damage. For purposes of the system study, therefore, it was assumed that trash would be allowed to enter the storage facility for later manual removal. The outlet from this facility, which would operate at flows much below storm flow rates, could be screened to exclude trash from the treatment works and downstream areas without creating appreciable maintenance problems.

An important aspect of water quality control along a natural watercourse is the protection of downstream uses through maintenance of minimum flows. In the case of Wilde Lake, it was considered desirable to protect existing stream bed ecology as well as maintain evaporation losses in Wilde Lake along with the release requirement imposed on the Wilde Lake dam by the Maryland Department of Water Resources. These factors will be taken into account in the development of operating criteria in a later section.

(2) Sedimentation. The operation of the storage facility as a sedimentation basin is of primary interest in sizing the facility. Large quantities of suspended solids must be removed from storm water in order to meet the minimum water quality standards described in Section II. D. In order to accomplish this objective, a reservoir may be operated as a combination storage and sedimentation facility, or a storage facility may be followed by a separate sedimentation device. Sedimentation will occur whenever the velocity of the storm flow is reduced by directing it into a structure or basin of large cross section, compared to the stream channel. Since the storage reservoir will function as a sedimentation basin regardless of its design criteria, the first approach was to analyze the effectiveness of this type of sedimentation device.

The unhindered settling velocity of free and discrete particles suspended in a liquid has been given as:

$$v_s = \frac{2g}{C_D} \frac{\rho_s - \rho}{\rho} \frac{V}{A_c}^{*} \qquad \text{(II-28)}$$

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*Fair, G. M., and J. C. Geyer, Water Supply and Waste-Water Disposal, Wiley, New York, 1954, p. 587.

where C_D is the Newtonian drag coefficient, a function of particle geometry, fluid characteristics, and Reynolds number, and where:

V = particle volume

A_c = projected area of the particle

g = acceleration of gravity

ρ_s = mass density of the particle

ρ = mass density of the fluid

If a velocity, v_o is defined for a specific ideal sedimentation chamber in the following terms:

$$v_o = \frac{Q}{A} \qquad \text{(II-29)}$$

where:

Q = overflow rate

A = surface area of chamber

then it can be shown that:

when $v_s > v_o$, the particle settles; and

when $v_s < v_o$, the particle does not settle.*

Since increasing the area, A, tends to reduce v_o, the fraction of particles having settling velocities such that they settle in a given chamber increases as A increases. Assuming that construction cost decreases with the volume of the sedimentation chamber, a reasonable design objective would be a chamber having large surface area, but relatively shallow depth. Such a design might create problems with sludge storage and removal, as discussed below.

When the characteristics and size distribution of the suspended particles are known, design of a suitable sedimentation facility is a straightforward problem. In the case of storm water, this information is not known and probably varies widely with time. Published reports (Refs. 30, 31, 33, 35, 37) and limited sampling have indicated the presence of colloidal and near-colloidal ($v_s \rightarrow 0$) solids in significant quantities. Hazen has calculated the settling velocity of various particles representative of those found in storm water as shown in Table II-21.

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*Hazen, Allen, "On Sedimentation," Trans. Am. Soc. Civil Engineers, Volume 53, 1904, p. 63.

TABLE II-21. SETTLING VELOCITIES OF SELECTED PARTICLES, AFTER HAZEN*

Kind of Material	Particle Diameter (μ)	Settling Rate (mm/sec)
Coarse sand	1000	10.0
Coarse sand	200	2.1
Fine sand	100	0.8
Fine sand	60	0.38
Fine sand	40	0.21
Silt	10	0.015
Coarse clay	1	0.00015
Fine clay	0.1	0.0000015

Since the distribution of particle sizes is not known, a conservative approach to sedimentation facility design is indicated. Hydrologic investigations reported in Section II.C estimated the mean runoff rate from sub-watershed number 14 at approximately 29.5 gallons per minute (gpm). A range of mean overflow rates might be considered in design of the sedimentation facility, but obviously none can be less than the mean runoff rate. Since reducing the overflow rate tends to reduce the area requirement for a given settling velocity (Equation II-29), the following table represents the minimum areas where settlement can be achieved under ideal conditions for the particles indicated.

TABLE II-22. MINIMUM SEDIMENTATION BASIN AREA REQUIREMENTS FOR SELECTED PARTICLES, SUB-WATERSHED NUMBER 14

Kind of Material	Particle Diameter (μ)	Minimum Area Requirement (square feet)
Coarse sand	1000	0.20
Fine sand	100	2.50
Silt	10	133.56
Coarse clay	1	13,356
Fine clay	0.1	1,335,600

The total land area in sub-watershed number 14 is 69 acres, or about 30,000,000 square feet. Inspection of Table II-22 indicates that design of plain sedimentation facilities for removal of particles significantly smaller than 10 microns is clearly not feasible.

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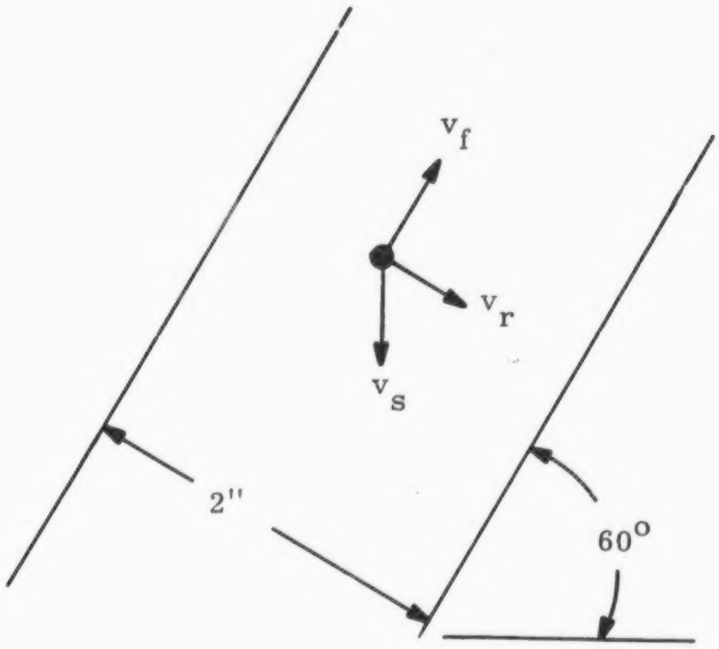
*Hazen, Allen, "Water Supply," American Civil Engineer's Handbook, Merriman, T. and T.H. Wiggin, Editors, Wiley:New York, 1930, p. 1474.

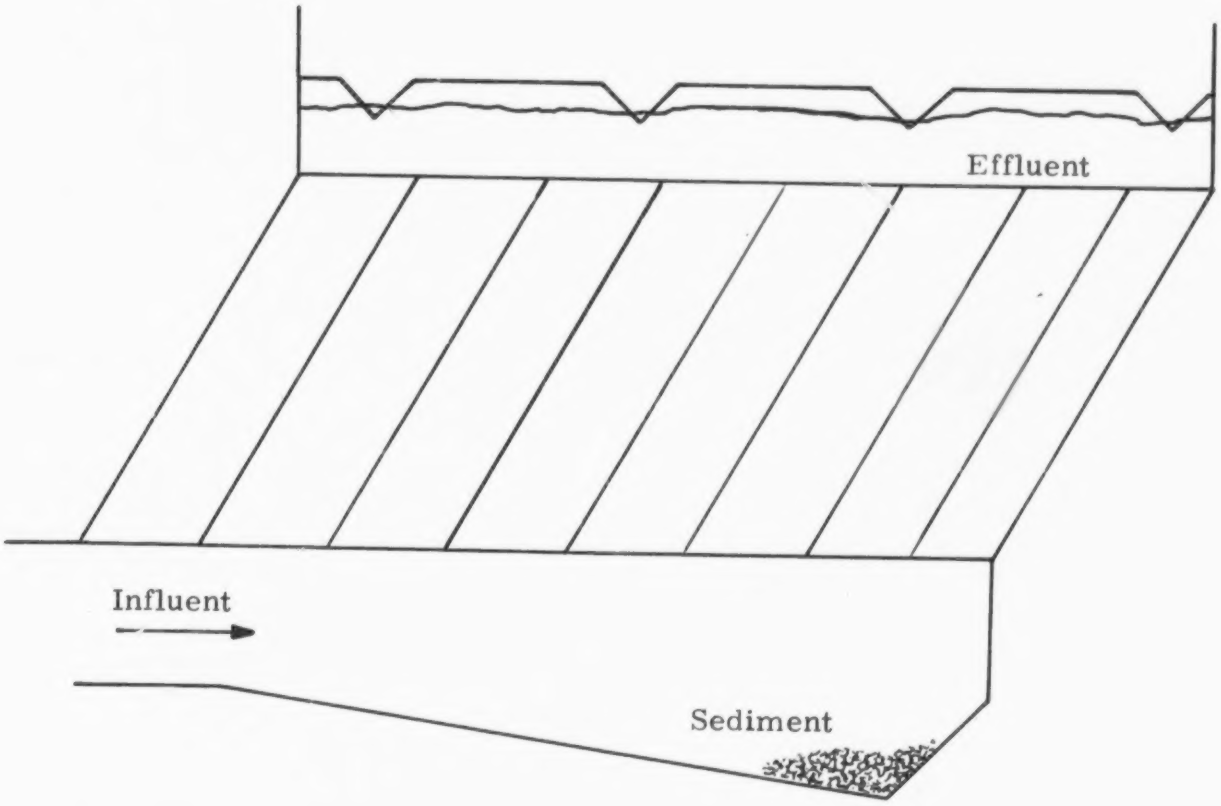
Even should the area requirements be compatible with the application, the effect of the highly nonuniform inflow on the operation of the basin must be examined. The problem of designing a single basin which can accept water at storm rates without creating an unfavorable velocity gradient across the cross-section is a very severe one, at best. The volume of water which must be accepted within a short period is large enough to induce large changes in the average water depth within the basin. This change alone would be sufficient to upset the sedimentation process, resuspending solids previously settled and causing turbulent conditions which would hinder settling of particles still in suspension. For these reasons, the use of a sedimentation device downstream from the storage reservoir was selected as the only feasible method of accomplishing both functions. The sedimentation device can be operated with its inflow equal to its outflow and will be subject to the area requirements shown in Table II-20. The reservoir can be sized on the basis of other criteria and operated on a fill-and-draw basis to supply the sedimentation device.

(3) Pretreatment Unit. Consideration must be given to the design of the sedimentation device since the required area to effect substantial removal of solids by plain sedimentation appears to preclude the use of a conventional sedimentation basin. Alternatives include the use of specialized sedimentation devices offering high effective-to-total surface area ratios, the application of chemical coagulants and coagulant aids to agglomerate small or light particles, and combinations of these techniques. Examination of settling requirements for particles below 1.0 micron in diameter led to selection of a combination approach - the use of an advanced sedimentation technique with chemical coagulants and/or coagulant aids. This facility, combined with chlorination of settled water, is referred to in later sections as the "Pretreatment" process. All runoff captured by the storage facility is subjected to this process, whether for subsequent final treatment or discharge to the stream.

Costs for the pretreatment phase were based on the use of a tube settler as marketed by Neptune-Microfloc, Inc., and described by Hansen and Culp (Ref. 46). This device employs a series of small diameter plastic tubes inclined upward, approximately 60 degrees from the horizontal. The flocculated water is permitted to pass upward through the tubes at controlled flow rates, typically between 3.0 and 5.0 gpm/square foot of cross-sectional area. Figure II-36 illustrates the principle of operation of the tube settler. Water moving up the tube at velocity v_f contains particles having a settling velocity v_s. The resultant velocity, v_r, tends to move the particles toward the tube wall, where they become trapped in a layer of particles previously removed. A two inch diameter tube four feet in length can be operated at flow velocities as high as 2.84 gpm/square foot tube area and still permit a 10 micron silt particle to move completely across the tube. The corresponding loading on a conventional sedimentation basin is 0.015 cm/second or 0.221 gpm overflow rate per square foot of surface area (Equation II-29)(see Table II-21). This comparison is based on ideal conditions and does not take into account hindered settling conditions caused by the concentration of particles as they near the sludge collection zone. In the case of plain sedimentation, basins are commonly maintained at a minimum depth of five to eight feet to allow sufficient space above the sludge storage zone for hindered settling to occur in a low velocity boundary zone between the top layer, where flow is close to average overflow rate, and the static



Vector Relationships for Particle in Tube Settler


Effluent
Influent
Sediment

Typical Operation of the Tube Settler

Figure II-36. Tube Settler - Principles of Operation

bottom layer. In the case of the tube settler, the steep inclination of the tubes causes the sludge to counterflow along the side of the tube as it accumulates, falling into a sump below the tube assembly. The tube settler configuration also requires influent and effluent plenums to distribute the flow to the tubes and to collect it after clarification. Figure II-37 shows a comparison of the volume of typical sedimentation facilities designed to remove 20 micron silt particles at various overflow rates. The tube settler volumes include provisions for influent flume, influent plenum, sludge storage sump, and effluent plenum, while the conventional sedimentation volumes are based on an average depth of 5.0 feet to provide for sludge collection and storage. Inside partitions and supporting structures are included, but outside walls are not.

Other sedimentation devices, such as tray settlers, were investigated. Performance characteristics fell between conventional sedimentation and the tube settler, but sediment removal and maintenance problems were more severe than either of the two techniques compared above.

c. Physical and Aesthetic Considerations. Inherent in the concept of the Local Storage, Treatment, and Reuse of Storm Water system is the close proximity of the system elements to the properties which benefit from the system. In addition to engineering and economic considerations then, it is essential that the physical appearance of the facilities be consistent with the immediate environment. In the case of Columbia, the storage and pretreatment units must be located in the flood plains to permit gravity flow through the units. In general, the flood plains of the Wilde Lake watershed have been dedicated to public use as open space. They are typically lightly forested with some shrubbery and underbrush and are bounded on all sides by residential land uses. Any construction within these areas must be particularly unobtrusive in order to achieve acceptability within the community. The pretreatment units and constructed storage can be placed below grade and any necessary superstructure, i.e., hatchways, vents, etc., can be disguised with shrubbery, fences, etc. In the case of open pond storage, the design of the pond must insure that its appearance under all conditions of operation will be one of an attractive, natural pond. All construction must be confined to an area which will permit at least a fringe of trees along the perimeter, thus avoiding a conspicuous discontinuity in the open space. Subsequent cost estimates of constructed storage and pretreatment units are based on their installation entirely below grade, while open pond estimates provide for design features to insure their acceptability.

d. Sizing Storage for Multiple Purposes. Earlier paragraphs described methods for determining required storage capacities for various conditions of water demand. It was found that for a given demand with a known proportion of lawn sprinkling uses, the storage requirement is a function of the reliability of the system or, stated differently, a function of the quantity of water drawn from the existing public water supply system in times of dry weather. Water quality considerations were found to affect operation policy more than design size, since use of the reservoir as a sedimentation device was found to be impractical. The application of the pretreatment step following storage does, however, impose a different type of yield requirement on the storage facility, in that it must be provided with a constant flow for proper operation, although that flow need not be

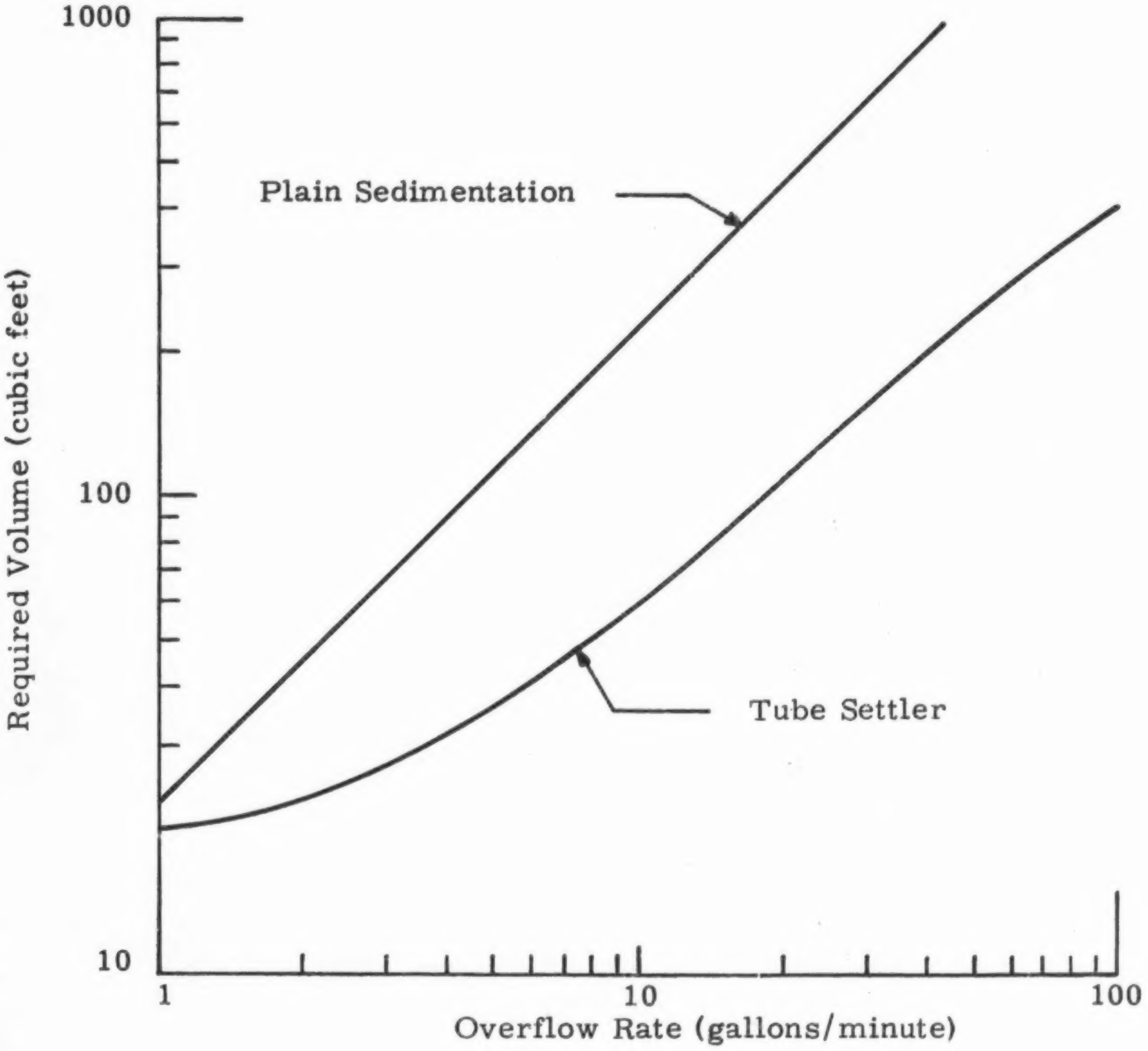


Figure II-37. Required Volume vs. Overflow Rate for Removal of 10 Micron Silt Particle Under Ideal Conditions

continuous. Physical and aesthetic considerations were examined and found to apply primarily to design criteria other than size, except that available land may limit the maximum attainable storage in a particular location.

Since the operation of the pretreatment unit appears to place a definable size requirement on the reservoir, that requirement will be evaluated first, then reviewed in the light of other requirements. Based upon previous discussions, the following assumptions are made with respect to the operation of the storage facility as a supply to the pretreatment unit:

(1) The reservoir will be operated on a fill-and-draw basis, storage capacity calculated as the difference between minimum and maximum pools.

(2) The highest proportion of total runoff feasible must be actually intercepted by the reservoir and subjected to pretreatment.

(3) When outflow to the pretreatment unit occurs, it must be at a constant controlled rate.

(4) Outflow need not occur at all times, e.g., intermittent operation of the pretreatment unit is permissible.

The proportion of the total runoff which is captured by the storage facility is a function of the volume of individual storms and the probability of available storage coincident with the occurrence of each storm. Since the reservoir is on a fill-and-draw cycle, the probability of available storage is a function of both the maximum storage capacity and the draw rate, or the rate of operation of the pretreatment unit. The analysis of reservoir performance will be in terms of recurrence intervals of storms and it will be initially assumed that storage is available to capture the entire volume of some design storm. When storms of larger volume or longer return interval occur, the excess quantity will be spilled, bypassing the pretreatment unit. Figure II-38 shows rainfall-intensity-duration curves for storms of various recurrence intervals in Howard County, Maryland. From the definition of recurrence interval, the following identity exists:

$$E(n) = \frac{P}{T} \qquad \text{(II-30)}$$

where:

$E(n)$ = expected number of storms meeting or exceeding conditions for a return interval T (years) occurring during period P (years)

Rainfall-intensity-duration curves can be used to generate runoff-intensity-duration curves for specific watersheds, using the methodology outlined in Section II.C.6. By application of probability and expectation theory to these curves, the quantity of runoff occurring in excess of the storm volume corresponding to any selected return interval may be calculated. Table II-23 lists the results of one such analysis.

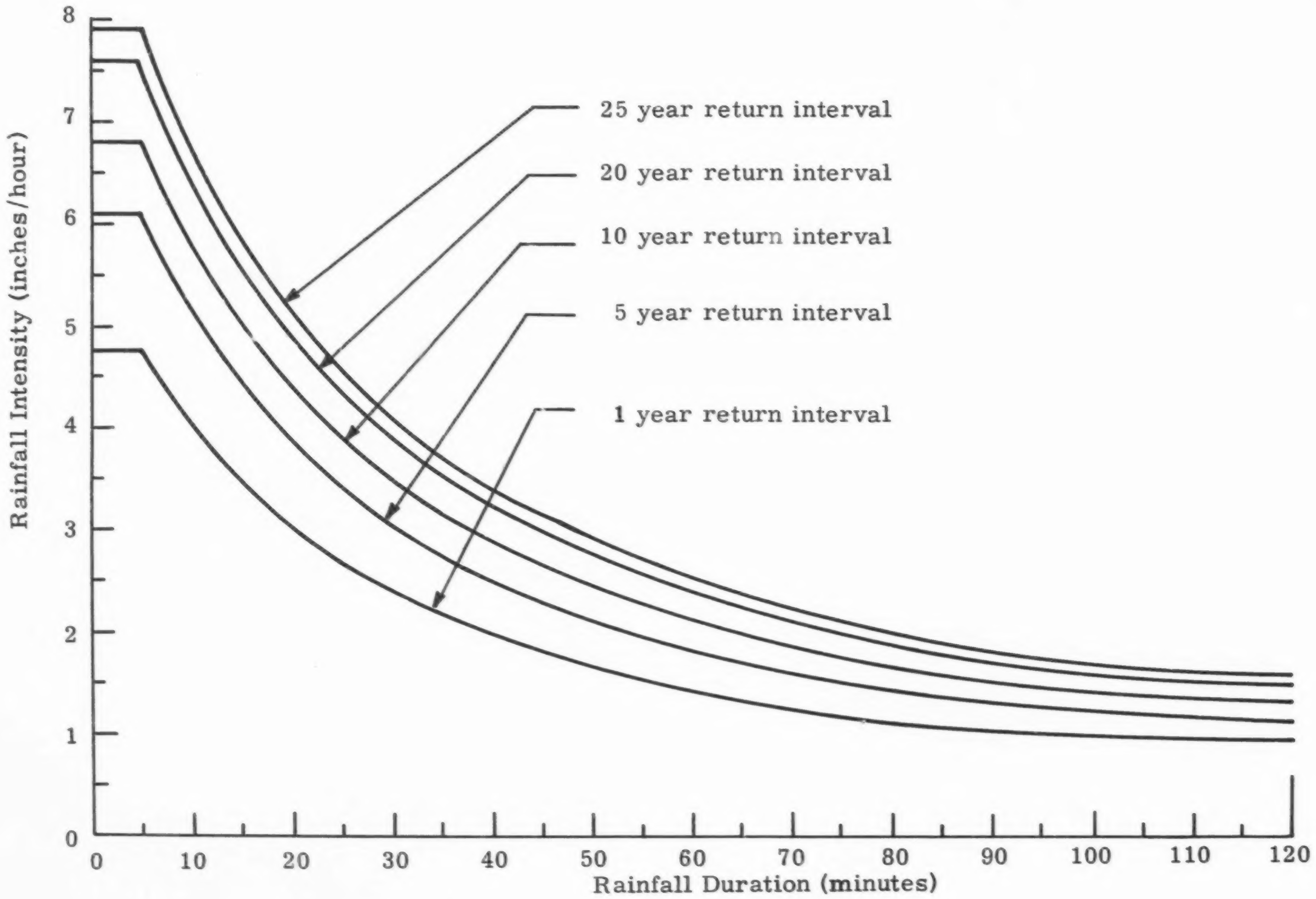


Figure II-38. Rainfall Intensity-Frequency-Duration Curves
Howard County, Maryland

TABLE II-23. EXPECTED QUANTITY OF RUNOFF IN EXCESS OF SELECTED STORM VOLUMES

(Imperviousness = 0.30, m = (lag time) = 4 minutes)
(Total annual runoff = 220,307 gallons)

Base Storm Return Interval (years)	Expected Excess Runoff (gallons/acre)	Excess (%)
1	16,160	7.34
5	5,050	2.29
10	3,066	1.39
20	1,857	0.84
25	1,584	0.72
50	963	0.44
100	579	0.26

Based on the most conservative calculation of storm volumes and on intensity-frequency-duration curves calculated to be conservative with respect to long-term record, a one year storm volume storage capability can be expected to capture at least 92.66 percent (100-7.34) of the total runoff in a year. This value is dependent, however, on the availability of maximum storage at the beginning of each storm, a function of the release rate of the reservoir.

The release rate can be as low as the annual mean daily runoff rate, or $\bar{q}$. When this rate is assumed with a storage capacity equal to the volume of a one year storm, analysis of the five year synthetic hydrograph for sub-watershed number 6 reveals that an additional 3.01 percent of the runoff can be expected to spill as a result of the occurrence of storms whose volumes, although less than the one year storm, exceed available storage. When the release rate is increased to $2.0\bar{q}$, the additional percentage of spill falls to 1.42 percent. Figures II-39 and II-40 show the approximate relationships between various releases, or pretreatment rates and spill percentages for a range storage capacity. It can be seen from these figures that a release rate of approximately $1.7\bar{q}$ in combination with a storage facility having a capacity equal to the volume of a one year storm will result in approximately nine percent of the runoff being spilled without treatment. The remaining 91 percent will pass through the pretreatment unit at a controlled rate. Lowering pretreatment rate to approximately $1.1\bar{q}$ increases the spill to 10 percent and greatly reduces storage requirements following the pretreatment unit where reuse is being practiced.

It is important to note that the water which does spill from the storage facility does so after passing through the storage structure where it has been detained for sufficient time to settle the larger and heavier solids. In view of the high quality of the balance of the water after pretreatment and the very infrequent nature of the spills, it was felt that restricting spills to 10 percent of total runoff was a realistic limit on the design and operation of the storage/pretreatment system. It would appear

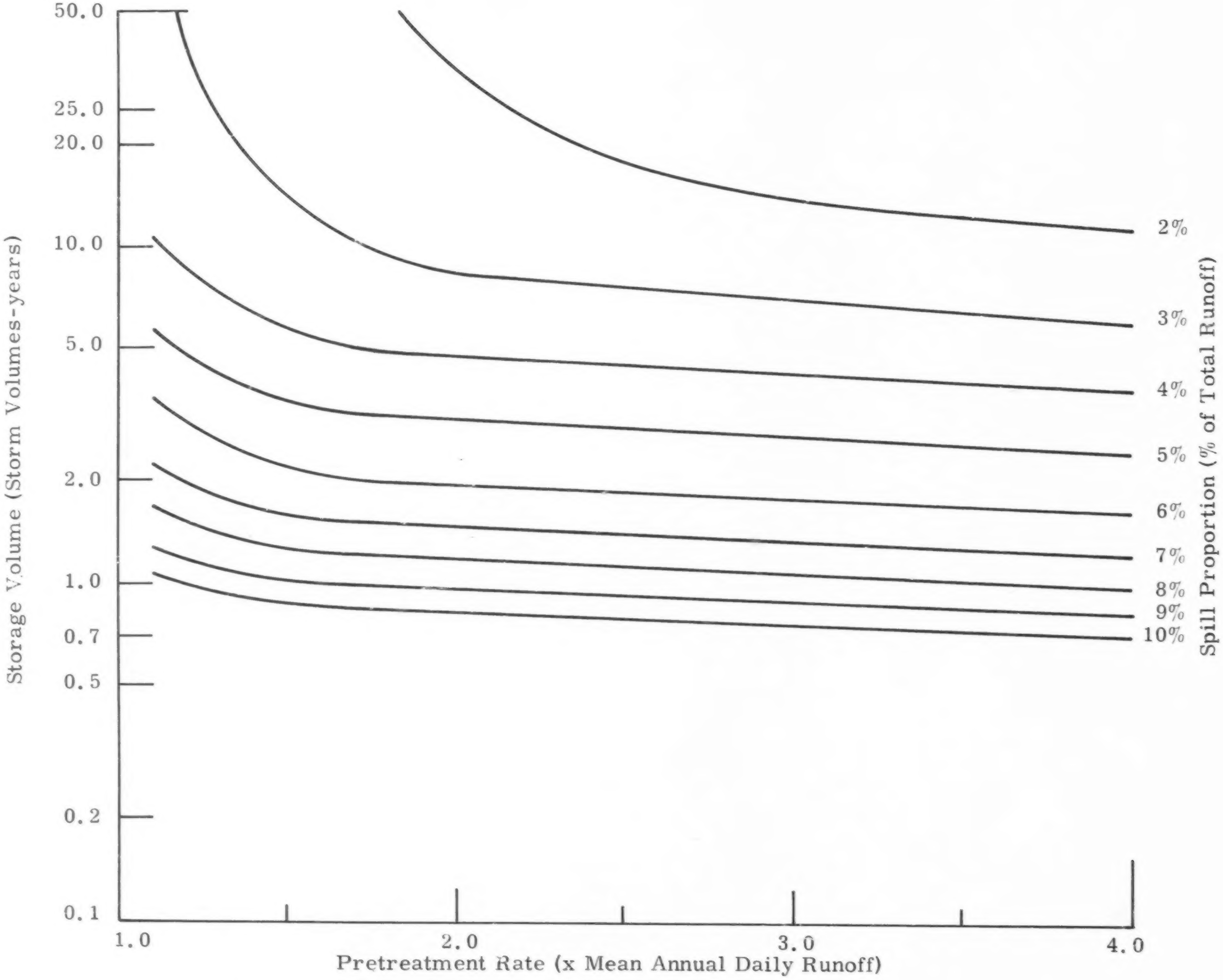


Figure II-39. Storage/Pretreatment Rate Combinations vs. Spill Percentages

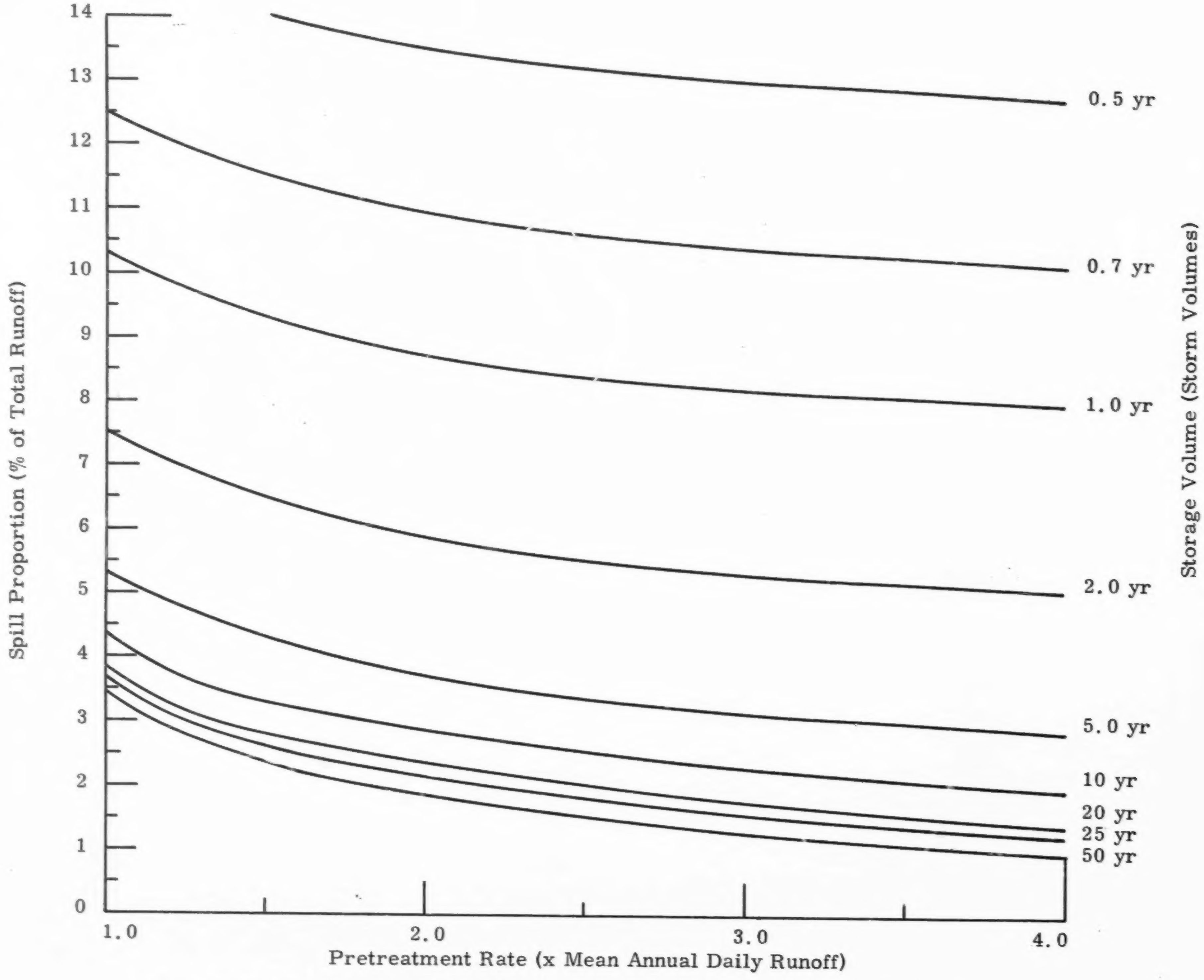


Figure II. 40. Pretreatment Rate/Spill Percentage Relationships vs. Storage then, that a reservoir sized at the one year storm volume operated in conjunction with a pretreatment unit controlled at a release rate somewhat greater than the mean runoff rate would process storm water within the water quality requirements of the system. A tradeoff obviously exists between spill volume and design criteria. Furthermore, a system having a given spill performance consists of a series of complex tradeoffs between storage capacity, release rate, and intermediate storage following pretreatment. Analysis of any of these tradeoffs, however, would require a quantitative estimate of the value of various levels of water quality, a task outside the scope of this study. Inherent in the study, however, is a method for inputing a value for water pollution control due to the study of the application of conventional methods of pollution control to the same watershed, reported in Chapter V. Both studies were directed to the same water quality requirement and the 10 percent maximum spill requirement was made of both systems. The method employed to estimate spills was also common to both studies.

Having determined the storage requirement for effective water pollution control, this requirement must be weighed against other reservoir purposes. Figure II-41 shows the storage/yield characteristics of sub-watershed number 16 plotted against the storage equivalent to the one year storm volume. It can be seen that the maximum long term demand of 25,327 gallons/day (mean runoff less 10 percent spill) can be sustained at the 80 percent reliability level. Similar relationships were found for each of the other sub-watersheds indicating that the water quality requirements on reservoir size result in a satisfactory reservoir from the water supply standpoint. The only other requirements found on reservoir size were those related to available land area and aesthetic considerations. Since these can only be determined by examination of a particular site with a particular size and construction type of proposed reservoir in mind, they were not considered until after the systems analysis effort was complete and conceptual designs were being produced from the results of the analysis. Further discussion of these points can be found in Chapters III and IV.

As a result of a comparison of the various influences on reservoir and pretreatment unit design, the following criteria have been selected as adequate to protect all uses under consideration:

(1) Net storage capacity of the reservoir shall be equal to the maximum volume for the sub-watershed or combination of sub-watersheds of a storm just meeting the one year return interval intensity.

(2) Release from the reservoir into the pretreatment unit will be controlled at a rate between 1.2 and 2.0 times the annual mean runoff rate.

(3) The pretreatment unit will be capable of operating at 2.0 times the annual mean runoff rate.

(4) No storage need be provided following pretreatment.

With respect to Criteria 2 above, it is noted that no discussion has been devoted to the effect of base flow on these criteria. Due to the use of the

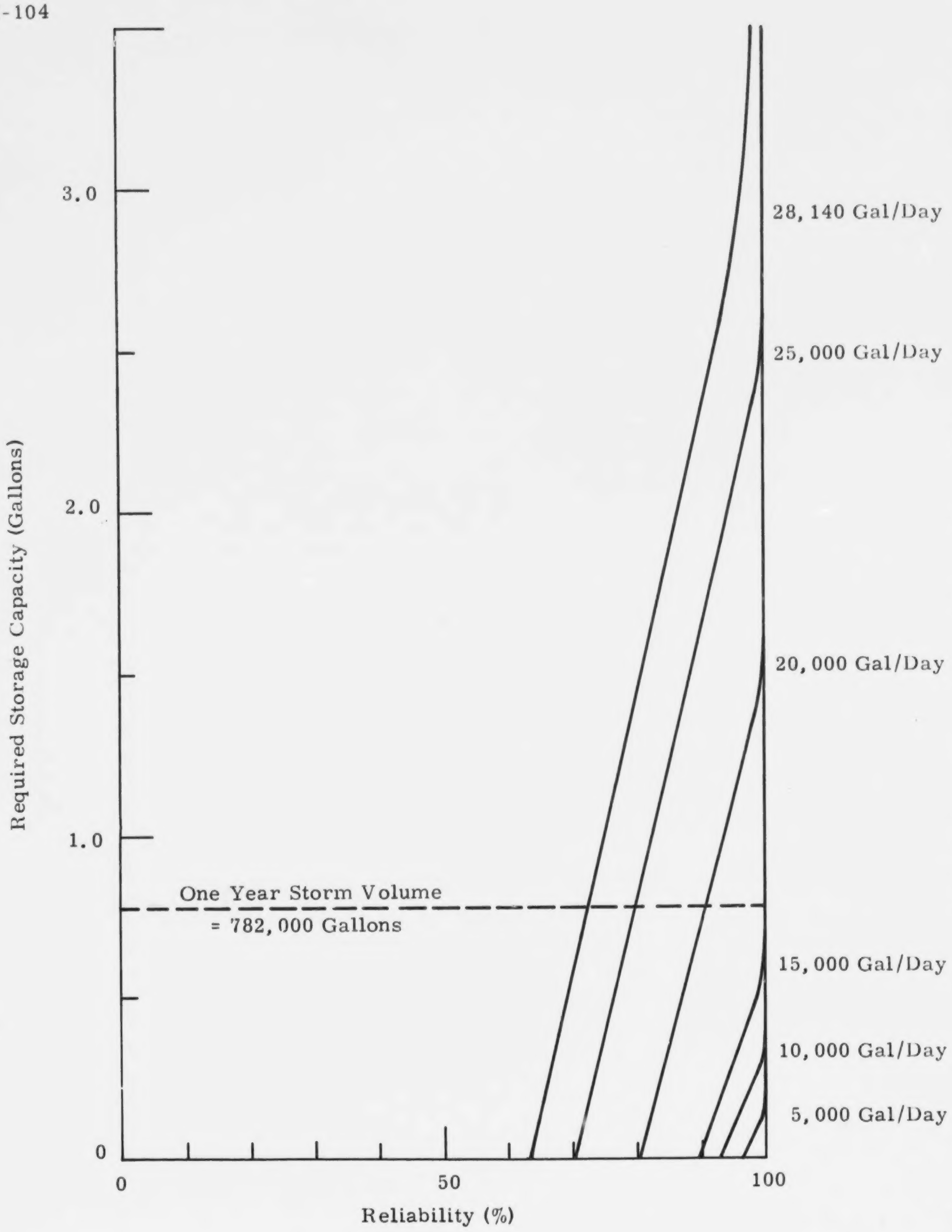


Figure II-41. Storage/Reliability Characteristics vs. Yield - Sub-watershed #16

natural flood plains as collecting storm drains, it is likely that a significant amount of relatively constant flow will occur as a result of springs and seepage from the ground. This quantity is in excess of the volumes of runoff described in the analysis, although there is no ready way of predicting its quantity. Due to the steady nature of the flow, it places no particular requirement on storage, but will increase the required release rate and pretreatment rate by an amount equal to its flow rate. It is considered that in an area such as Columbia, the base flow would probably not be more than 30 percent of the mean runoff rate from storm water. Hence the requirement of 2.0 times the mean runoff rate for pretreatment design.

4. Construction Costs

Construction costs were calculated for the five types of storm water storage reservoir listed in Table II-16. All types except the open ponds have an alternate application as treated water storage facilities and the same construction costs were employed for both applications. In addition, other types of storage were considered for treated water only, namely, standpipes, elevated tanks, and hydropneumatic tanks. Table II-24 lists all types of storage considered for either application. This section will describe the methods employed to compute parametric costs for each of these storage types. The following section will outline operation and maintenance costs for the storm water application only. Similar costs for treated water storage applications can be found in Section II.G.4.

TABLE II-24. RESERVOIR CONSTRUCTION TYPES

Construction Type	Application Storm Water	 Treated Water
Natural Storage		
Open Pond	X	
Ground Level Storage		
Steel Tank - Above Grade	X	X
Steel Tank - Below Grade	X	X
Concrete Tank - Above Grade	X	X
Concrete Tank - Below Grade	X	X
Elevated Storage		
Standpipe		X
Elevated Tank		X
Pressure Storage		
Hydropneumatic Tank		X

Construction costs for storage facilities were considered to be made up of the following:

(a) Land costs, applicable wherever the purchase of the land is a necessary cost of the system.

(b) Site preparation costs, including clearing and grubbing, rough grading, finish grading, and seeding or sodding after construction.

(c) Burial costs, including excavation and backfill for tanks which are installed below grade.

(d) Erection costs, which include foundations, tank erection, painting of metal parts, and in the case of ponds, necessary excavation, fill, embankment, spillway construction, and bottom and slope preparation.

In each case, actual estimates were made of construction costs for a number of hypothetical installations of various types and sizes in the Columbia area. Prices of 1967 to 1968 were used and allowances made for engineering, inspection, and construction contingencies. The estimates were then plotted against the appropriate size parameter and a smooth curve extended between the points. This curve was then replaced by a series of linear approximations which closely followed it and the inflection points reduced to a table of sizes and costs. This table was input to the computer in the systems analysis, so that the cost of any size facility could be estimated by a linear interpolation between two points on the table. In general, the original estimates were made at sizes which permitted the curve to be well defined in the range of possible application in the systems analysis.

a. Land Costs. In most applications, land can be purchased for a per-acre price which is relatively uniform throughout the project area. Methods for computing the required land area for each type and size of tank or pond were developed and costs were determined by simply permitting the computer program to multiply the land area in acres by the cost in dollars per acre, which was input separately. Computation of required land area for open ponds required the conceptual siting of a variety of pond sizes in the Wilde Lake watershed and the computation of the land area. In every case, a 30 foot border was provided beyond the high water line to protect slopes and permit decorative fringing with trees and shrubbery. No attempt was made to force areas into regular shapes except for ease of computation. Figure II-42 shows the resulting land area/capacity curve, with the linear approximation overlaid. Table II-25 lists the inflection points of the linear approximation which were input to the computer for pond land area computations. All other types of storage were treated as cylindrical tanks and their land requirements were computed on the basis of height/diameter (H/D) ratios. These ratios, found by dividing tank height by tank diameter, were determined by plotting H/D for standard designs of a variety of tanks. Figure II-43 shows such a plot for ground level tanks and elevated tanks. Figure II-44 shows H/D's for standpipes. The corresponding linear approximations can also be seen on these figures and Tables II-26, II-27, and II-28 list the ratios used in the computer program. In each case, the tank diameter is found by the formula:

$$\mathrm{DIAM} = \left(\frac{0.61702\mathrm{CTAN}}{\mathrm{H/D}}\right)^{0.3333} \qquad \text{(II-31)}$$

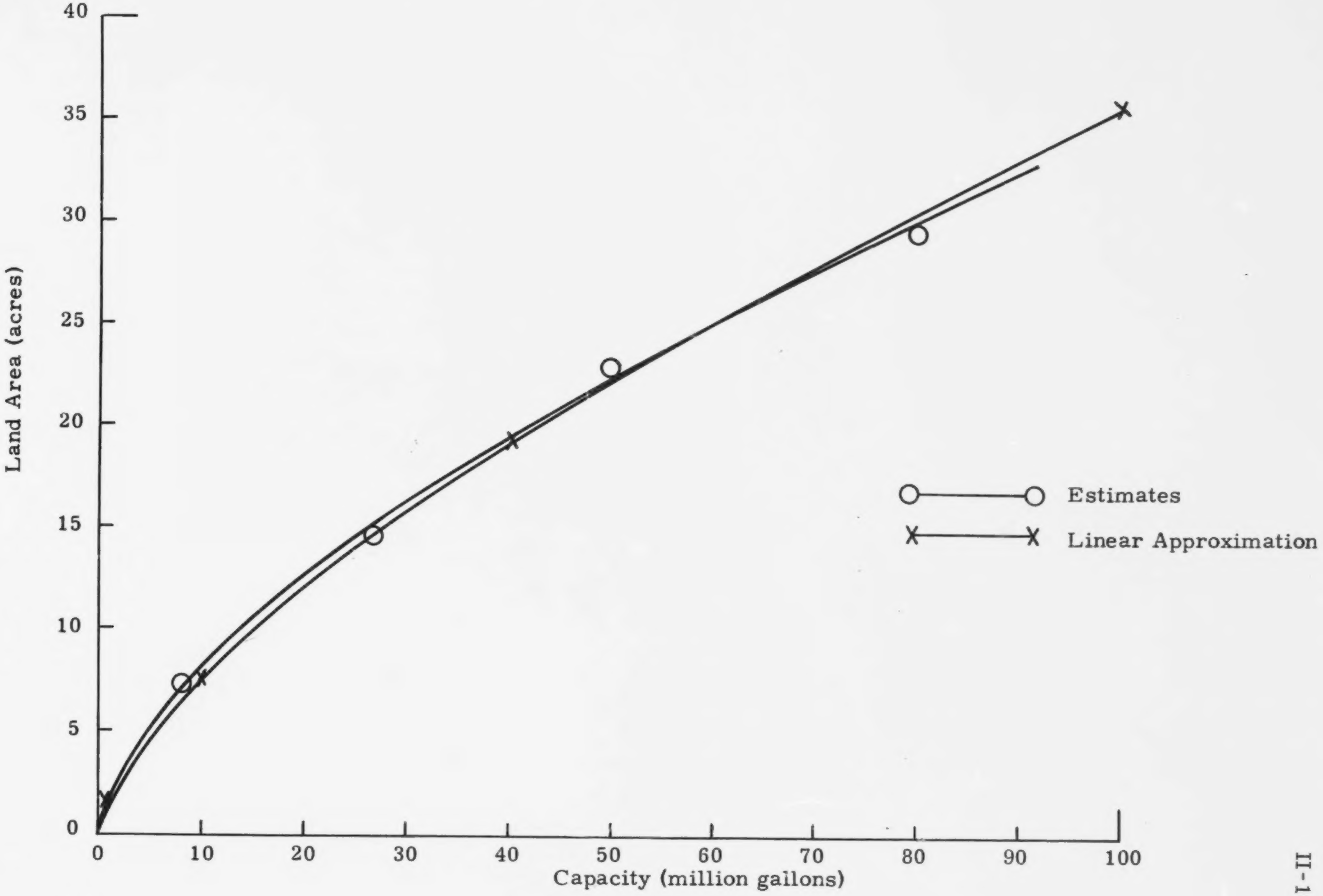


Figure II-42. Land Area vs. Capacity - Open Ponds

TABLE II-25. LAND AREA VS. CAPACITY - OPEN PONDS

Capacity (gallons)	Land Area (acres)
0.0	0.0714
10,000	0.0714
100,000	0.340
1,000,000	1.609
4,000,000	4.110
10,000,000	7.645
20,000,000	12.250
40,000,000	19.550
100,000,000	36.370

TABLE II-26. H/D VS. CAPACITY - GROUND LEVEL TANKS

Storage Capacity (gallons)	H/D
0.0	1.000
50,000	0.900
200,000	0.759
500,000	0.640
1,000,000	0.512
2,000,000	0.366
3,000,000	0.285
5,000,000	0.210
7,000,000	0.180
10,000,000	0.165
100,000,000	0.0348

TABLE II-27. H/D VS. CAPACITY - ELEVATED TANKS

Storage Capacity (gallons)	H/D
0.0	0.995
250,000	0.995
300,000	0.900
500,000	0.710
700,000	0.588
1,000,000	0.490
1,250,000	0.442
1,500,000	0.410

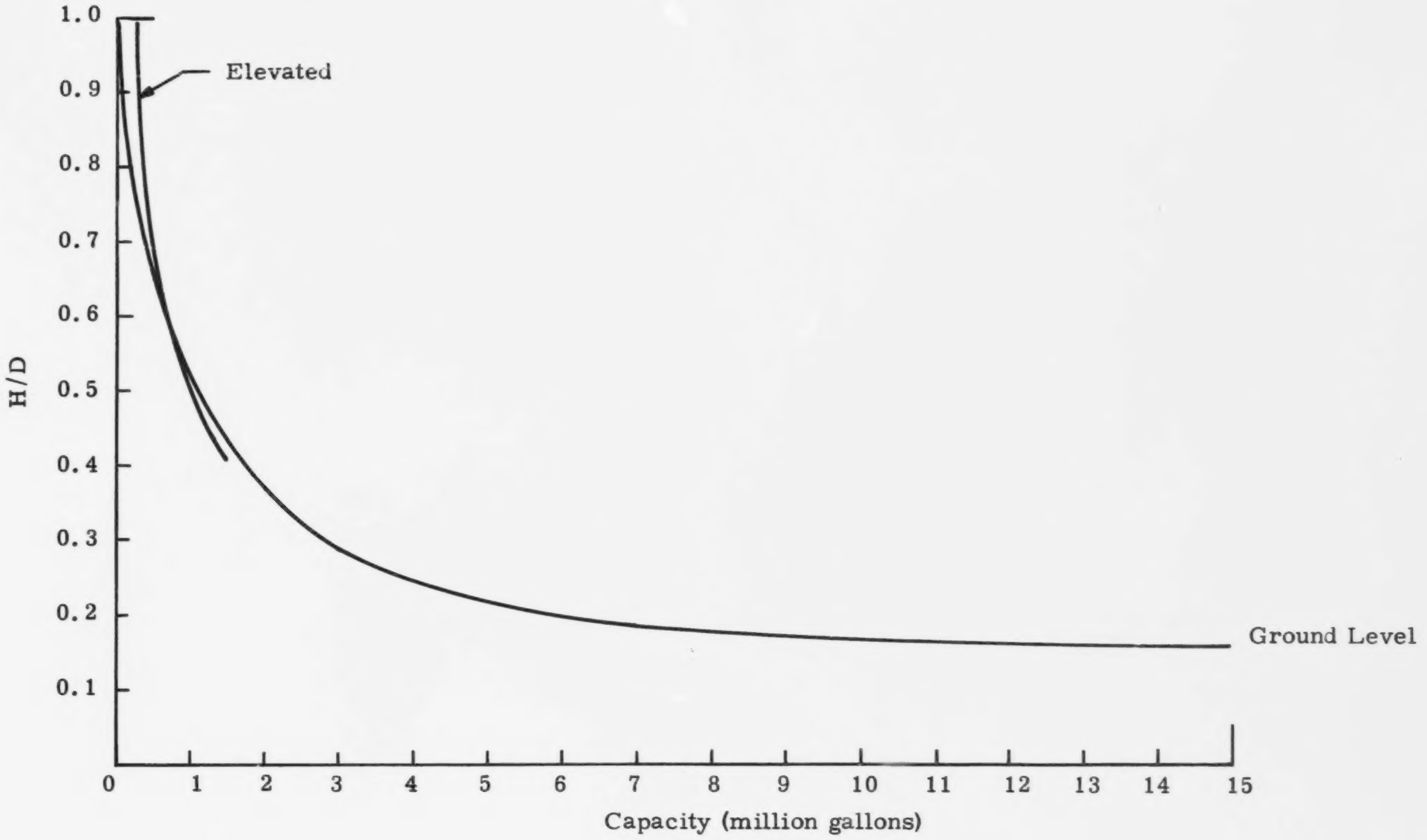


Figure II-43. H/D vs. Capacity - Ground Level and Elevated Tanks

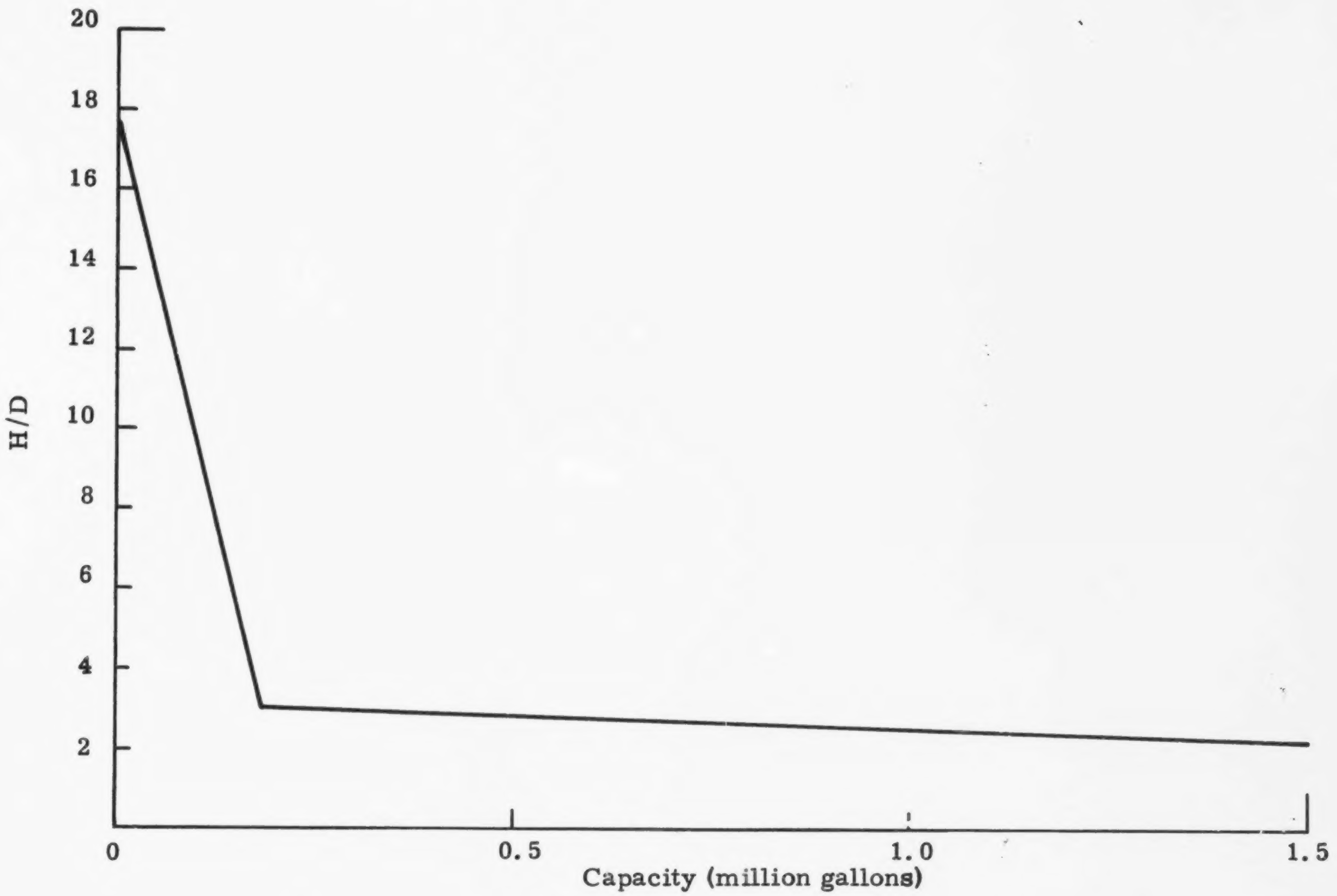


Figure II-44. H/D vs. Capacity - Standpipes

TABLE II-28. H/D VS. CAPACITY - STANDPIPES

Capacity (gallons)	H/D
0.0	17.25
180,000	3.00
1,500,000	2.20

where:

CTAN = tank capacity, gallons

H/D = ratio of tank height to diameter

DIAM = tank diameter, feet

Required land area is found by adding 30 feet to the diameter, thus providing for a minimum 15 foot border around the tank. This calculation is performed as follows:

$$BAR = 0.00002296\ (DIAM + 30.0)^2 \qquad (II\text{-}32)$$

where:

DIAM = tank diameter, feet

BAR = required land area, acres

Since the land to be used in the Wilde Lake watershed was entirely public open space, the initial systems analysis computer runs were made with land cost set at zero. Should the value of land to be used for system construction become a significant consideration, insertion of the appropriate per-acre cost into the computer input will result in revised estimates.

b. Site Preparation Costs. Since the costs treated under this heading are primarily related to land area and not to type of construction, they were evaluated on a per-acre basis and applied to all tanks. The single exception to this is the case of open ponds, where site preparation costs were considered as part of erection costs, and included in that tabulation. Figure II-45 shows the curve plotted from estimates and the linear approximation. The table of values used in the computer program is shown as Table II-29. Land area is computed as described in the paragraph above on land costs. Per-acre site preparation costs corresponding to various capacities are obtained by linear interpolation of the table. The interpolated value is then multiplied by the land area to yield total site preparation cost.

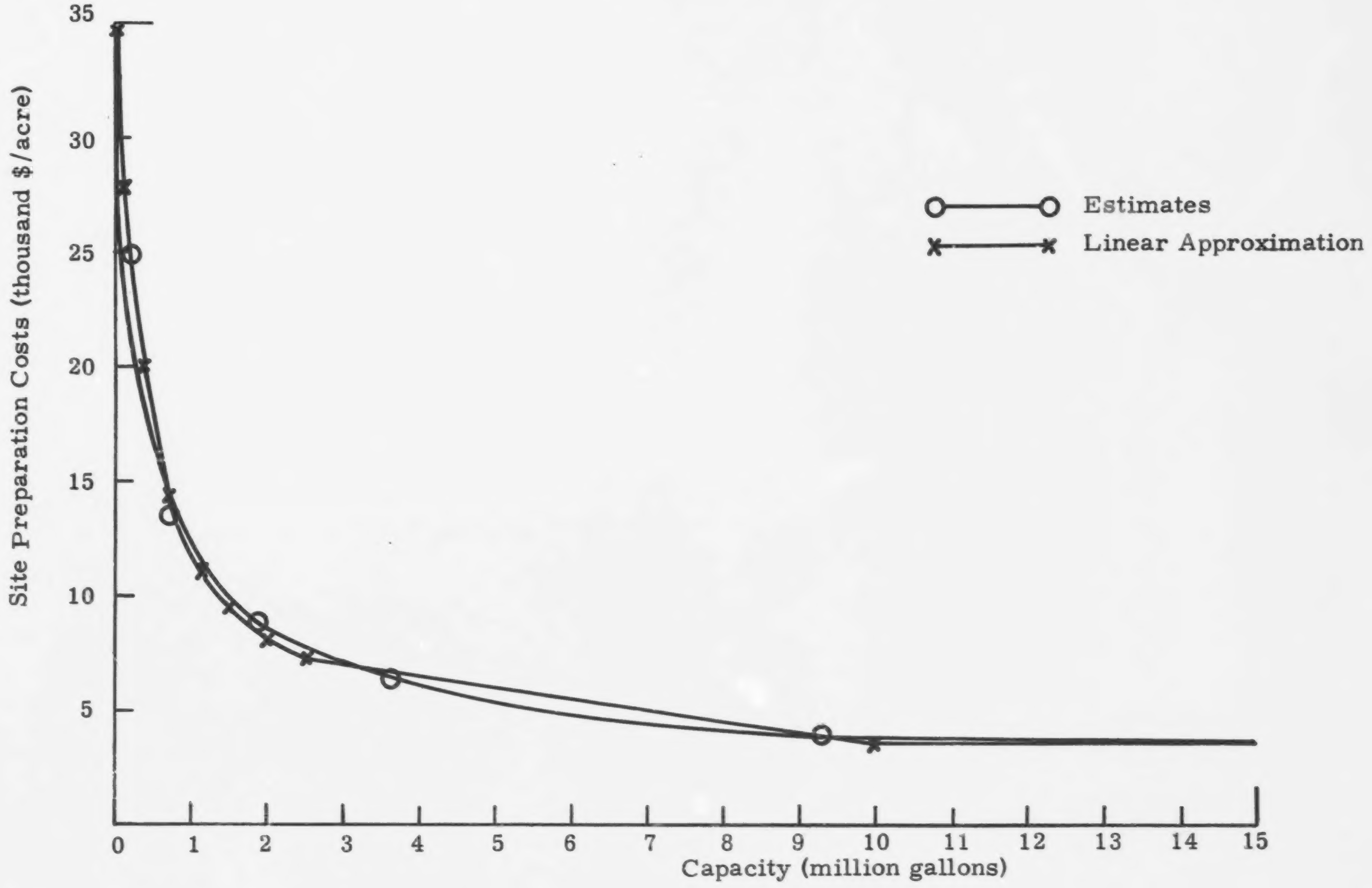


Figure II-45. Site Preparation Costs vs. Capacity - Storage Tanks

TABLE II-29. SITE PREPARATION COSTS VS. CAPACITY

Capacity (gallons)	Site Preparation Costs ($/acre)
0.0	34,500
100,000	27,750
300,000	20,300
700,000	14,200
1,150,000	11,000
1,500,000	9,500
2,000,000	8,200
2,500,000	7,400
10,000,000	3,400
100,000,000	3,400

c. Burial Costs. Burial costs are computed only for the ground level storage tanks installed below grade. In these cases, the total cost of excavation and backfill associated with the construction of various underground tanks was estimated and the results plotted as Figure II-46. This figure also shows the linear approximation of the curve which is represented by Table II-30. The tank capacity is used to interpolate between points on the table, yielding burial costs in dollars per gallon. When this figure is multiplied by the capacity, the total burial costs are obtained.

TABLE II-30. BURIAL COSTS VS. CAPACITY

Capacity (gallons)	Burial Costs ($/gallon)
0.0	0.0510
100,000	0.0480
500,000	0.0390
1,000,000	0.0310
1,500,000	0.0269
2,000,000	0.0230
2,500,000	0.0206
3,000,000	0.0188
3,500,000	0.0176
4,500,000	0.0162
10,000,000	0.0134
100,000,000	0.0134

d. Erection Costs. Estimates were obtained from tank fabricators and erectors for in-place costs of a variety of tank types and sizes. Table II-31 is a summary of that information. Figure II-47 shows curves for various types of above grade and elevated tanks. Tanks designed for burial must be constructed somewhat differently to account for the additional stresses, and information from fabricators indicated an approximate 15

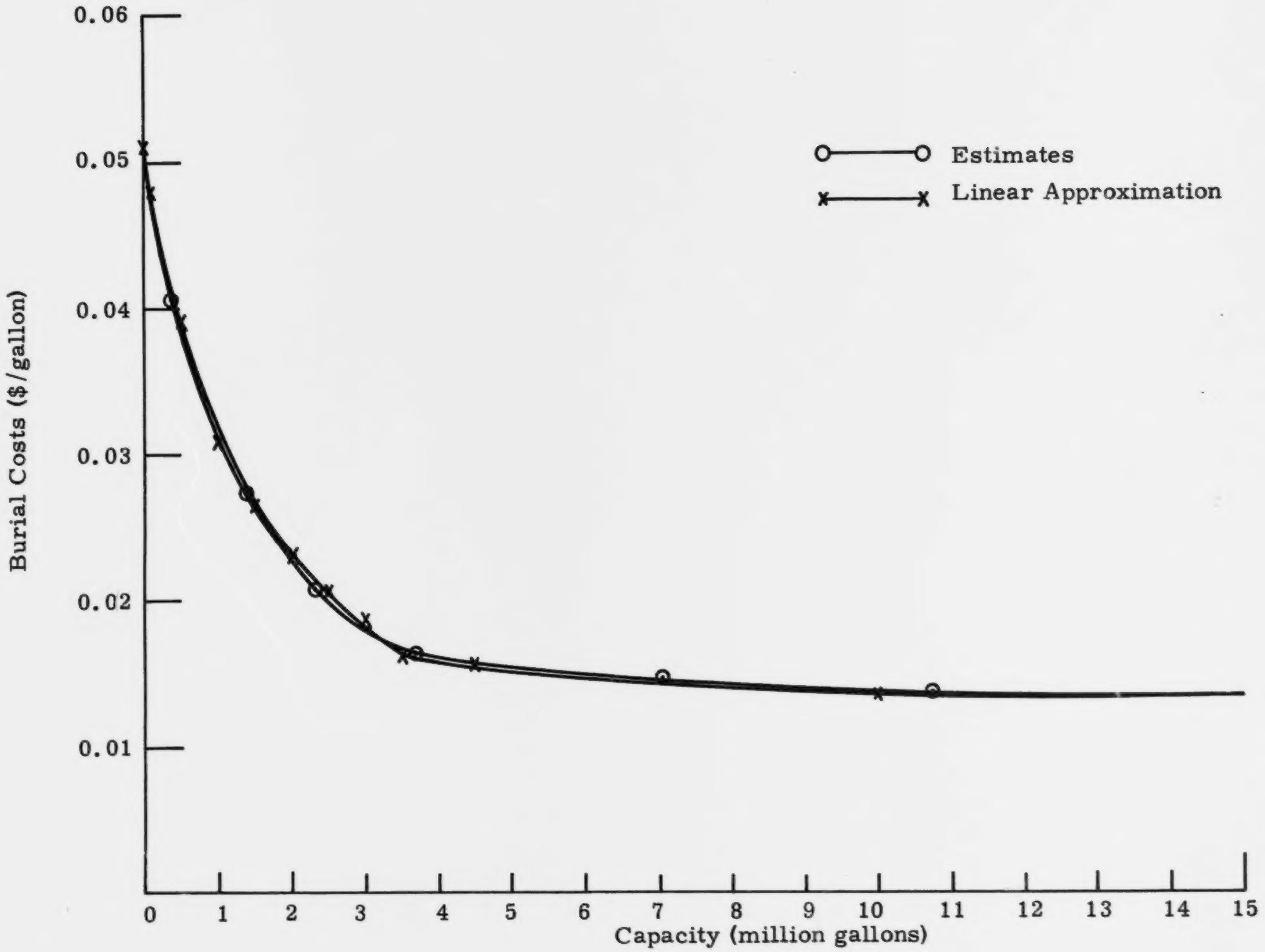


Figure II-46. Burial Costs vs. Capacity - Storage Tanks

TABLE II-31. SUMMARY OF TANK ERECTION COST ESTIMATES

	Capacity (gallons)	Erection Costs ($)
Ground Level Tanks - Steel	50,000	10,000(1)
		8,400(2)
	100,000	13,000(1)
		11,000(2)
	200,000	15,500(2)
	500,000	30,000(1)
		27,000(2)
	1,000,000	53,000(1)
		44,000(2)
	2,000,000	110,000(1)
		77,000(2)
	5,000,000	175,000(2)
	10,000,000	340,000(2)
Ground Level Tanks-Concrete	100,000	18,500(3)
	250,000	28,000(3)
		45,000(4)
	500,000	45,000(3)
		68,000(4)
	750,000	80,000(4)
	1,000,000	69,000(3)
		100,000(4)
	2,000,000	125,000(3)
		150,000(4)
	5,000,000	250,000(3)
		325,000(4)
	10,000,000	500,000(3)
		500,000(4)
Elevated Tanks	50,000	38,000(1)
		30,000 ⟷ 33,000(2)
	100,000	45,000(1)
		38,500 ⟷ 44,000(2)
	200,000	60,000(1)
		42,500 ⟷ 52,500(2)
	500,000	110,000(1)
		85,000 ⟷ 95,000(2)
	1,000,000	220,000(1)
		170,000 ⟷ 186,000(2)
	2,000,000	375,000(1)
		290,000 ⟷ 330,000(2)
Standpipes	100,000	19,000(1)
	500,000	45,000(1)
	1,000,000	80,000(1)
	2,000,000	130,000(1)

(1) Pittsburgh-DesMoines Steel Co.
(2) Chicago Bridge and Iron
(3) The Crom Co.
(4) The Preload Co.

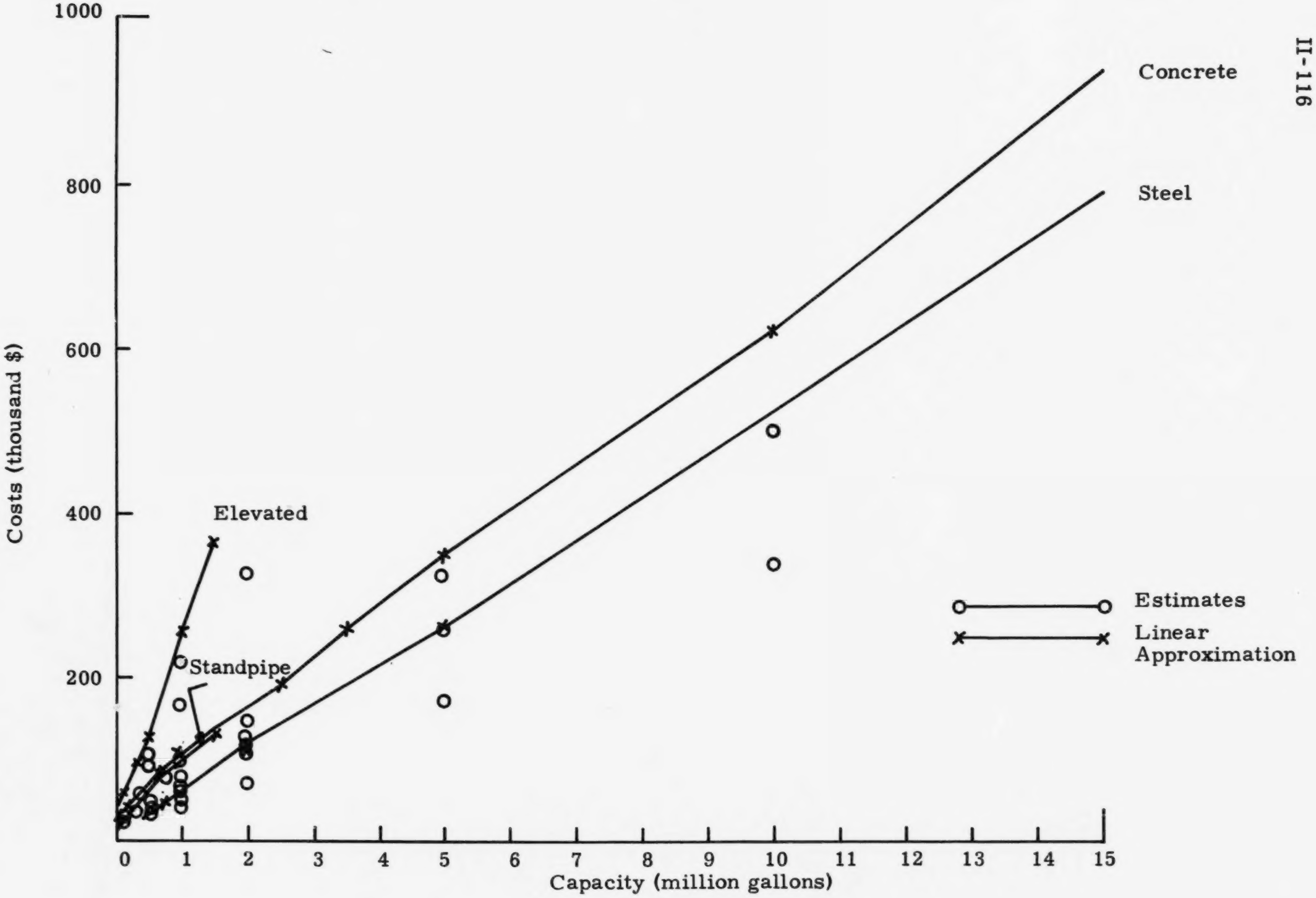


Figure II-47. Erection Costs vs. Capacity - Concrete, Steel, and Elevated Tanks and Standpipe

percent increase in the tank cost. When the burial option is selected in the program, then 15 percent is added to the tank erection cost in addition to the computation of the burial cost. Table II-32 lists the program inputs used to describe the erection cost capacity relationships for various tank types.

In the case of open ponds, the erection cost is highly site-dependent. Furthermore, a tradeoff exists between excavation of the site to increase storage and providing additional spillway elevation. In addition to being dependent upon the relative costs of excavation and spillway and embankment construction, the optimum combination is a function of the site topography and the spillway-embankment width. In order to arrive at an approximate parametric relationship between pond capacity and cost, a site was selected from the Wilde Lake watershed (sub-watersheds numbers 4 and 5) and a dam location determined. Using a large scale topographic map with one foot contours, pond capacity was determined for each foot of elevation of the dam up to the maximum feasible elevation at the site. These capacities were those of completely natural ponds with no excavation. Next, capacities were recalculated for each elevation with excavation of the basin until the incremental cost of excavation (in dollars per gallon) equalled the incremental cost of the next change in dam elevation. These final capacities were used to develop the cost versus capacity curve which appears in Figure II-48. In general, excavation accounted for about one-third of the final pond volume. A summary of the cost estimates appears in Table II-33. Table II-34 is the final values used in the computer input.

5. Operation and Maintenance

In the preparation of the preliminary parametric cost data for use in the computer program, the operation and maintenance costs for the storm water storage facilities was considered to be under three headings. These were:

(a) Routine checks of storage and pretreatment facilities
(b) Trash removal from storage facilities
(c) Sediment removal from storage and pretreatment facilities

In this initial analysis, it was assumed that the routine checks of the facilities would be conducted approximately once per week and would require one-half man-hour per reservoir. It was assumed that trash removal would be required on half of the inspections and an additional two-tenths of a man-hour were allotted for these inspections.

In the generation of the parametric cost data, it was assumed that sediment would be removed by pumping from the tanks and pretreatment facilities using tank trucks. One hour was allotted for this operation requiring a semi-skilled laborer and an equipment operator. It was assumed that this operation would be required 25 times per year and that the tank truck rental would be $100 per day or $8.33 per hour or per operation.

For the disposal of the sediment, it was assumed that a satisfactory dumping site would be available within two miles of the reservoir. No allowance was made for the operation and maintenance of a sediment disposal facility in this study.

TABLE II-32. ERECTION COSTS VS. CAPACITY - STORAGE TANKS

	Capacity (gallons)	Erection Cost ($)
Ground Level Storage Steel	0.0	5,000
	10,000	5,000
	50,000	11,000
	200,000	21,000
	400,000	32,000
	700,000	48,000
	1,000,000	65,000
	2,000,000	122,000
	5,000,000	263,000
	100,000,000	5,260,000
Ground Level Storage Concrete	0.0	15,000
	10,000	15,000
	50,000	20,000
	100,000	25,000
	300,000	51,000
	900,000	100,000
	1,500,000	139,000
	2,500,000	197,000
	3,500,000	259,000
	5,000,000	355,000
	10,000,000	625,000
	100,000,000	6,250,000
Elevated Tanks	0.0	26,000
	10,000	26,000
	50,000	45,000
	100,000	55,000
	200,000	72,000
	350,000	102,000
	500,000	132,000
	1,000,000	258,000
	1,500,000	367,000
Standpipes	0.0	15,000
	10,000	15,000
	50,000	20,000
	200,000	32,000
	400,000	51,000
	700,000	82,000
	1,500,000	137,000

TABLE II-33. SUMMARY OF CONSTRUCTION COST ESTIMATES - OPEN PONDS
(Cost items in dollars)

Capacity (gal)	Concrete Yards	Excavation Yards	Pond Area (sq. ft.)	Concrete Cost	Embankment Cost	Excavation Cost	Clear & Grub	Rip Rap	Net Total	Cont. +25%	Gross Total
52,950	16.02	87.9	2,500	1,153	30	160	30	109	1,482	371	1,853
97,950	20.11	161.9	4,000	1,448	30	360	40	402	2,280	570	2,850
159,450	23.89	263.8	5,500	1,720	30	535	55	740	3,080	775	3,855
238,800	39.22	393.4	7,000	2,820	54	810	70	786	4,540	1,135	5,675
339,300	42.77	560.1	9,000	3,078	60	1,050	90	1,142	5,420	1,355	6,775
465,000	52.03	768.4	11,250	3,746	71	1,320	112	1,031	6,280	1,570	7,850
630,000	70.37	1,037.0	15,000	5,063	101	1,300	150	970	7,584	1,896	9,480
858,000	87.00	1,416.7	20,500	6,226	497	608	207	1,878	9,454	2,364	11,818
1,015,000	102.00	1,660.0	23,000	7,344	520	1,850	230	1,536	11,480	2,870	14,305
1,105,000	110.00	1,800.0	24,400	7,920	529	2,040	244	1,187	11,920	2,980	14,900
1,780,000	151.00	2,850.0	35,000	10,872	840	3,358	350	3,500	18,920	4,730	23,650

TABLE II-34. CONSTRUCTION COST VS. CAPACITY - OPEN PONDS

Capacity (gallons)	Construction Costs ($)
0.0	1,765
50,000	1,765
200,000	5,000
1,000,000	13,290
4,000,000	33,800
10,000,000	62,600
20,000,000	99,800

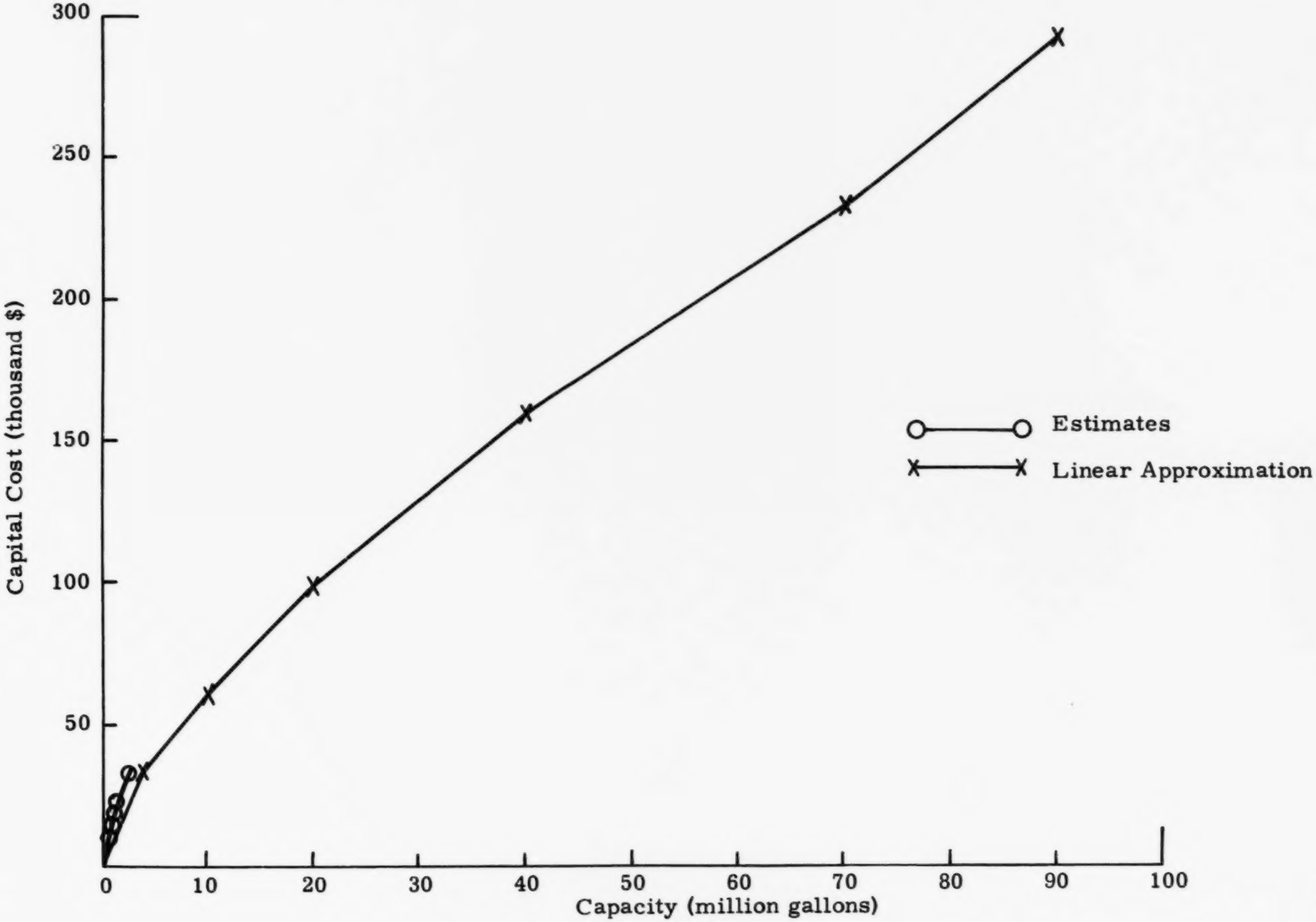


Figure II-48. Construction Costs vs. Capacity - Open Ponds

The labor rates used in this initial analysis were based on the use of semi-skilled laborers at the rate of $2. 50 per hour, plus 15 percent for fringe benefits, or $2. 88 per hour, for the routine checks, trash removal, and as a helper on the sediment removal operations. The labor rate for the equipment operator was estimated at $3. 50 per hour, plus 15 percent, or $4. 03 per hour.

Table II-35 summarizes the estimated cost of the routine checks and trash and sediment removal used in the development of the parametric cost data for the computer program. These costs are based on a storage facility receiving 10, 000, 000 gallons of runoff per year.

TABLE II-35. SUMMARY OF SEDIMENT REMOVAL COSTS

	Number of Occurrences per Year	Man-hours	Rate	Labor Cost	Equipment Cost	Per Occurrence	Total per Year
Routine Check	100	0. 5	2. 88	1. 44	0. 50	1. 94	97. 00
Trash Removal	25	0. 2	2. 88	0. 58	----	0. 58	14. 50
Sediment	25	1. 0	2. 88	2. 88	----	----	
Removal		1. 0	4. 03	4. 03	8. 33	11. 47	381. 00
Total trash and sediment removal, cost per year							$ 492. 50
Trash and sediment removal, cost per gallon runoff							$0. 00004925

The parametric cost data selected and used in the computer program was $0. 0005 per gallon of runoff for both tank type storage and open ponds, independent of size. As discussed later in Chapter IV, the operating and maintenance costs for the storage reservoirs and pretreatment unit were further refined in making the cost estimates for the various local storage and reuse system. In these estimates, the costs for sediment removal from the storage reservoirs and pretreatment unit were separately estimated and the frequency of the various operations and man-hour estimates were revised based on the actual design. These later operation and maintenance costs also include the cost of chemicals used in the pretreatment units and the labor and materials for preventative maintenance and repair.

In addition to the costs of the routine checks and cleaning of the storage reservoir and pretreatment unit, the parametric cost input included provision for the painting and repair of metal surfaces, hatchways, doors, etc. as well as repair to concrete structures. Ground maintenance was not included since the normal maintenance that would be given to the open space where the facilities are located is expected to be adequate. Minor repairs and painting can be accomplished as part of the routine inspections noted above. Concrete structures are not expected to require significant maintenance during the time span of the economic analysis. Metal tanks, however, must be periodically painted and repaired. Buried tanks require maintenance of their internal surface, while tanks installed above grade have two surfaces to be maintained. Repair and maintenance experiences with a number of tanks in the Baltimore area were supplied by Whitman, Requardt, and Associates, along with the number of square feet maintained in each case. The total cost, taking into account painting and repairing exposed metal surfaces, has been found to average \$0.05 per square foot per year.

The inside surface area of buried tanks was computed as follows:

$$\text{SURI} = 3.14159\ \text{DIAM}\ (\text{HIGH} + 0.5\ \text{DIAM}) \qquad \text{(II-33)}$$

where:

DIAM = tank diameter computed as in Equation II-31, feet

HIGH = tank height = tank diameter times H/D ratio, feet

SURI = total inside surface area, square feet

This area includes inside walls, floor, and ceiling. The outside area of above grade tanks was computed to include outside walls and roof as follows:

$$\text{SURO} = 3.14159\ \text{DIAM}\ (\text{HIGH} + 0.25\ \text{DIAM}) \qquad \text{(II-34)}$$

where:

SURO = total outside surface area, square feet

Total surface area of tanks above grade was taken as inside area plus outside area, or SURI + SURO. Maintenance costs are then computed by multiplying tank area by \$0.05 per square feet per year.

G. Treatment Methods

1. Treatment Requirements as a Function of Quality

Section II. E described the wide range of pollutants and pollutant concentrations found in storm water by other investigators. The literature searches reported in Sections II. D and II. E failed to locate reports of successful treatment of storm water immediately after collection from small watersheds. In the absence of a comprehensive storm water sampling and analysis program in the Columbia area, it must be assumed that storm water pollutants will be similar in number and concentration to those found in runoff from comparable areas. Table II-11 in Section II. E listed the pollutants whose presence is assumed and indicated the expected range of concentration of each. Specification of unit processes intended to achieve a given level of treatment must necessarily be conservative, due to the lack of reported field experience with similar problems. It is not considered that conventional water treatment plant experience is directly transferable to this problem, due to the extremely "random" nature of certain pollutants, notably suspended solids.

Section II. D described the four levels of treated water quality to be considered in the systems analysis. Various unit processes have been examined for each level, with attention to the maximum pollutant concentrations expected as well as the ability of the process to withstand severe shock loads. Table II-36 lists the processes selected, indicating the treatment levels for which each is used.

Algae control is applicable only to those cases where storm water will be stored in an open facility. The term is intended to include all treatment steps necessary to prevent degradation of the stored water or the creation of a nuisance condition. Under some conditions, this might include steps to prevent mosquito or other insect breeding as well as the control of aquatic growths. Further discussion of these considerations can be found in Section II. F. 3. After thorough analysis of the actual water to be treated, several of the processes listed might become unnecessary. Examples are pH adjustment and softening. Limited sampling conducted to date indicates that total hardness is low under base flow conditions and lower yet during storm flows, while pH varies between 6. 3 and 8. 3, depending on flow. Further sampling might indicate a need for some chemical adjustment of these parameters, however.

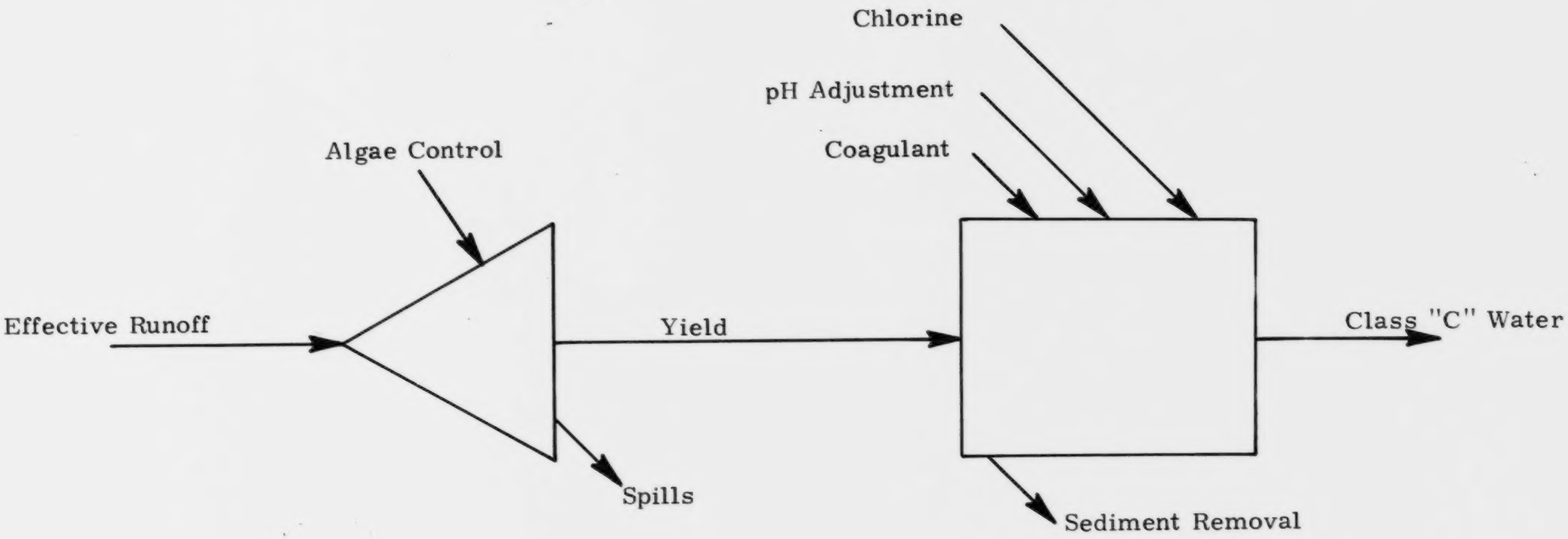
2. System Descriptions

a. Treatment to Class "C" Quality. Class "C" Treatment, outlined on Figure II-49, consists of mechanical trash screening, raw water storage with algae control, sedimentation, pH adjustment, and chlorination. Trash is excluded from the storm water collection system to a degree by the inlet design, which incorporates gratings and restricted openings. Smaller objects can enter the system, however, and parts of the subsequent collection system are open channels and natural watercourses, permitting further ingress of trash. Due to the difficulty of preventing a stoppage at the entrance to a storage facility, it is anticipated that no screening be provided at that point. This choice would depend on the difficulty of removing large objects from the storage basin itself.

TABLE II-36. PROPOSED TREATMENT PROCESSES

Unit Process Type	Water Use Category			
	A-A	A	B	C
Raw Water Storage with Algae Control	X	X	X	X
Aeration	X	X		
Sedimentation	X	X	X	X
Flocculation	X	X	X*	X*
Straining (with pre-coat)			X	
Sand Filtration	X	X		
pH Adjustment	X	X	X	X
Softening	X	X		
Chlorination	X	X	X	X
Carbon Adsorption	X			

*IP required


Algae Control
Effective Runoff
Yield
Spills
Chlorine
pH Adjustment
Coagulant
Class "C" Water
Sediment Removal

STORM WATER STORAGE

- Algae Control*

PRETREATMENT

- Flocculation
- Sedimentation
- pH Adjustment*
- Chlorination

*If required

Figure II-49. Class "C" Treatment System

The point of withdrawal from the storage facility would then be screened to exclude trash from subsequent treatment processes. Since this screen would not be required to operate at storm flows, routine maintenance, as discussed in Section II.F.5, should permit satisfactory operation without the installation of mechanical cleaning devices.

Raw water storage might consist of any type of structure or basin, the only requirement being that its maximum elevation be at or below inlet elevation. This will permit storm flows to enter by gravity without adversely affecting the hydraulics of the storm water collection system. The possibility of pumping storm water at storm flow rates was excluded from the analysis for reasons of reliability. All available equipment designed for such intermittent, high rate service is electric motor driven. The correlation between electric power outages and severe thunderstorms is considered relatively high in an area of this type and such a joint occurrence would result in spills of large quantities of untreated storm water. Even though the facility could be designed to avoid flood damage from such an occurrence, the overall reliability was considered inconsistent with the water quality objectives of the study. Storm water storage facilities actually considered are described in Section II.F.

The decision to employ a specialized sedimentation device, or pretreatment unit, effectively eliminated sedimentation considerations from the design of the storm water storage facility. The criteria which were employed in sizing this facility are described in detail in Section II.F.3. The storm water storage facility precedes the pretreatment unit and coagulants are introduced as the water is withdrawn from storage. The pretreatment unit is used as a point of application for pH adjustment chemicals, if required, and chlorine. Effluent from this combined operation is directed to the Class "C" water uses or allowed to overflow to the natural watercourse.

b. Treatment to Class "B" Quality. Class "B" treatment consists of all the unit operations of Class "C", namely storm water storage with algae control (if necessary) and pretreatment, followed by a straining operation (See Figure II-50). This operation is the Microstraining process of the Glenville Kennedy, Division of Crane Company, or equivalent. It consists of a fine mesh screen which is coated with an expendable filter media, or precoat, so that effective removal of all particles 10 microns and larger can be achieved. Additional chlorine is added after this process in order to maintain an adequate residual throughout the distribution system. Following straining, the effluent is available for Class "B" or "C" water uses. Excess water will be permitted to overflow after the pretreatment process before entering the straining equipment. In this manner, distributed water will be at least Class "B", while excess water will be at least Class "C".

c. Treatment of Class "A" Quality. Class "A" treatment, shown in Figure II-51, consists of the unit operations of Class "C", storm water storage with algae control (if necessary) and pretreatment, followed by aeration, additional chemical coagulation, and mixed media filtration. Chemical additions and filter designs will be selected to ensure clarification of the effluent to 1.0 Jackson Turbidity Unit or better. This effluent, after rechlorination, will be distributed to Class "A", "B", or "C" uses. Excess water will be permitted to overflow after the pretreatment unit.

d. Treatment to Class "AA" Quality. The treatment processes for Class "A" and Class "AA" are identical with the single addition of activated

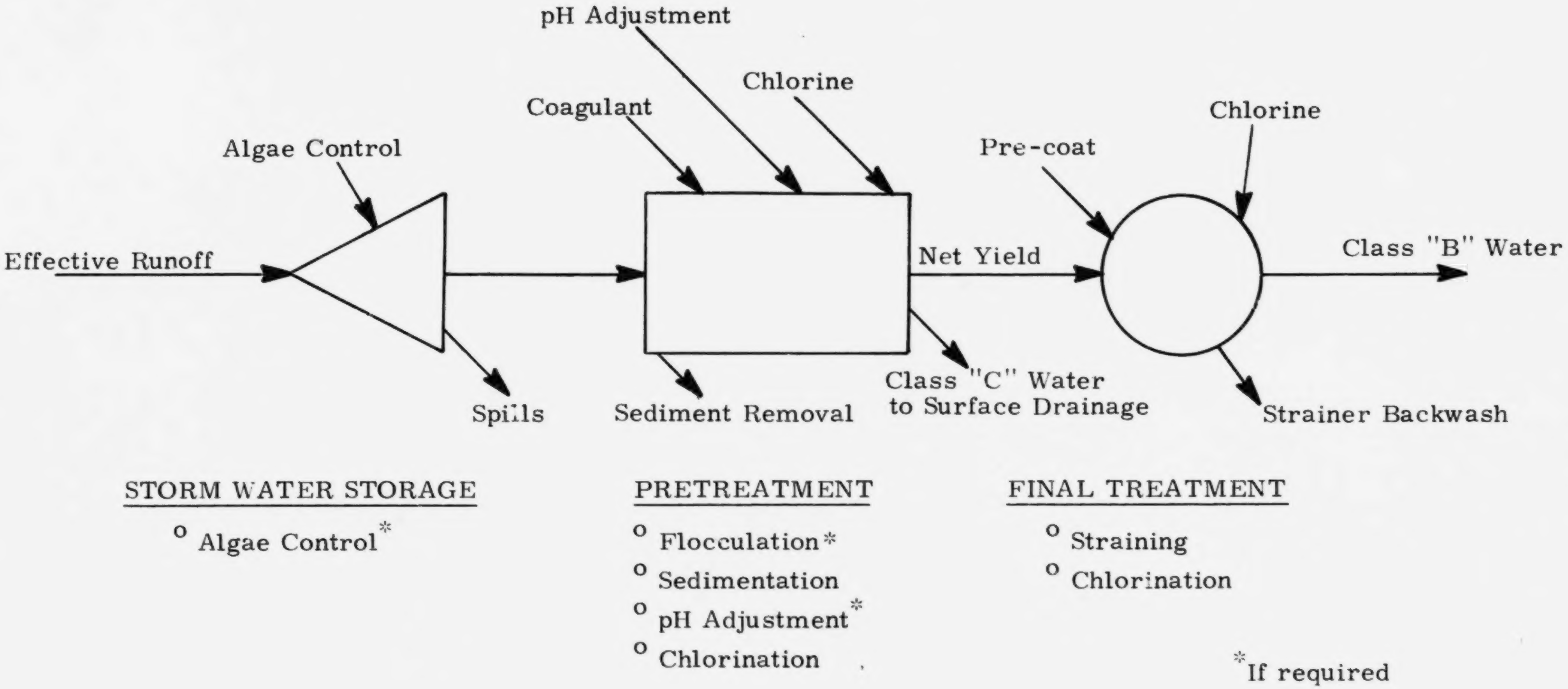
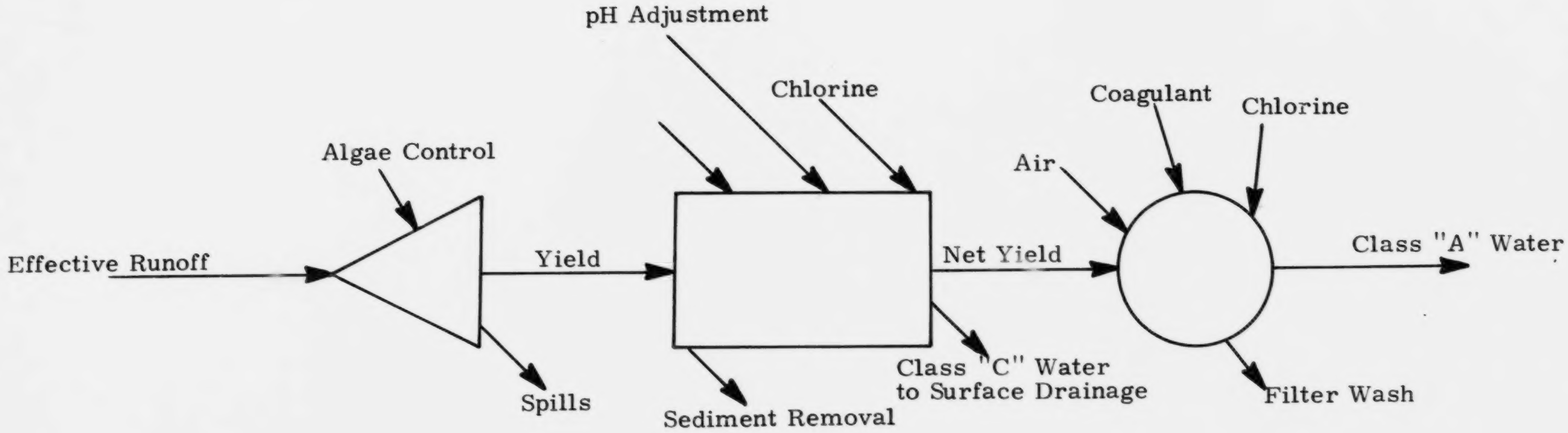

pH Adjustment
Chlorine
Coagulant
Chlorine
Algae Control
Pre-coat
Effective Runoff
Net Yield
Class "B" Water
Spills
Sediment Removal
Class "C" Water to Surface Drainage
Strainer Backwash
STORM WATER STORAGE
o Algae Control*
PRETREATMENT
o Flocculation*
o Sedimentation
o pH Adjustment*
o Chlorination
FINAL TREATMENT
o Straining
o Chlorination
*If required

Figure II-50. Class "B" Treatment System


pH Adjustment
Chlorine
Coagulant
Chlorine
Algae Control
Air
Effective Runoff
Yield
Net Yield
Class "A" Water
Spills
Sediment Removal
Class "C" Water to Surface Drainage
Filter Wash

STORM WATER STORAGE

- Algae Control*

PRETREATMENT

- Flocculation*
- Sedimentation
- pH Adjustment*
- Chlorination

FINAL TREATMENT

- Softening*
- Flocculation*
- Filtration
- Chlorination

*If Required

Figure II-51. Class "A" Treatment System

carbon adsorption in Class "AA" applications, as shown in Figure II-52. This process removes tastes and odors as well as providing a buffer against occasional excessive concentrations of adsorpable organics and other dissolved materials. An activated carbon column utilizing a replaceable charge would be employed. The carbon charge would be periodically removed and disposed of or regenerated at a central facility. The capacity of the carbon column would be set at a value substantially in excess of anticipated requirements, in the interest of long life and security. Rechlorination would be practiced following the carbon treatment. As before, excess water will overflow after the pretreatment step.

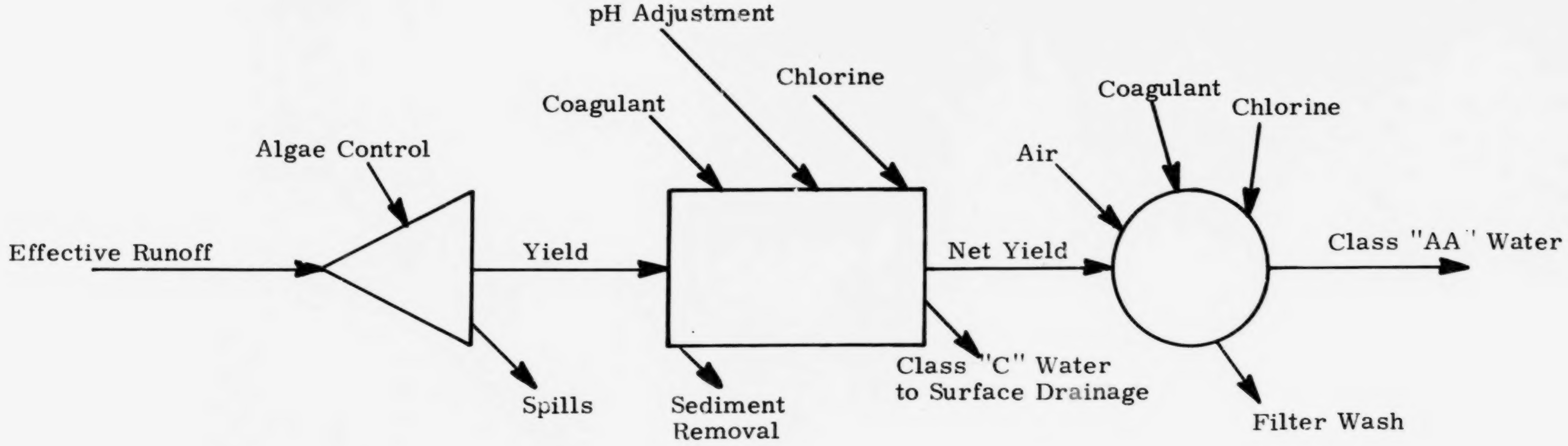
e. Summary. The foregoing paragraphs do not represent suggested design for every circumstance. Actual configurations should be determined after suitable investigation of the characteristics and variations in the quality of the storm water actually available. The processes described are only intended to be representative of adequate treatment under most local conditions and may well constitute over-design in many applications. Due to the supplemental nature of the water supply being evaluated, fail-safe controls can be employed to shut down the treatment facility in the event of malfunction without loss of supply to the connected demand. In this manner, considerable redundancy commonly a part of minicipal water plant design can be eliminated, and small package-type treatment units of low cost might be profitably employed. For purposes of the systems analysis, however, no innovations have been considered. Every process discussed above has known performance characteristics consistent with the most difficult expected requirements.

3. Equipment and Facility Costs

Construction costs for pretreatment, final treatment, pumping, and treated water storage were determined by computing estimated costs for each of a series of design capacities and assuming that a continuous function exists between the calculated points. In each case, a minimum capacity was determined, below which it was assumed that costs would not decrease with decreasing capacity. Underground construction within the flood plain area was assumed for pretreatment and final treatment units, both to minimize pumping costs and overflow provisions and to permit unobtrusive installations in residential neighborhoods. Underground, ground level, and elevated options were considered for treated water storage.

a. Pretreatment. Estimated construction costs for the pretreatment unit are listed in Table II-37. The pH adjustment process was not included in the parametric costs, due to its nominal cost and the high probability of a satisfactory pH at the Columbia location. Costs are based on a solution feed of both coagulant and chlorine, obtained from hypochlorite. Use of chlorine gas was also investigated, but not included in the parametric cost analysis, due to the wide range of capacities being investigated and the probable inapplicability of chlorine gas feed at the smaller sizes.

The complete pretreatment construction cost/capacity function was approximated by fitting a curve to the points developed in Table II-37. This curve was, in turn, approximated by a series of inflection points and straight lines, permitting the data to be input into a computer program as a table of cost/capacity relationships. This table is shown as Table II-38 and the two curves are illustrated on Figure II-53.


pH Adjustment
Chlorine
Coagulant
Coagulant
Chlorine
Algae Control
Air
Effective Runoff
Yield
Net Yield
Class "AA" Water
Spills
Sediment Removal
Class "C" Water to Surface Drainage
Filter Wash

STORM WATER STORAGE

- Algae Control*

PRETREATMENT

- Flocculation
- Sedimentation
- pH Adjustment
- Chlorination

FINAL TREATMENT

- Softening*
- Flocculation*
- Filtration
- Carbon Adsorption
- Chlorination

Figure II-52. Class "AA" Treatment System

*If Required

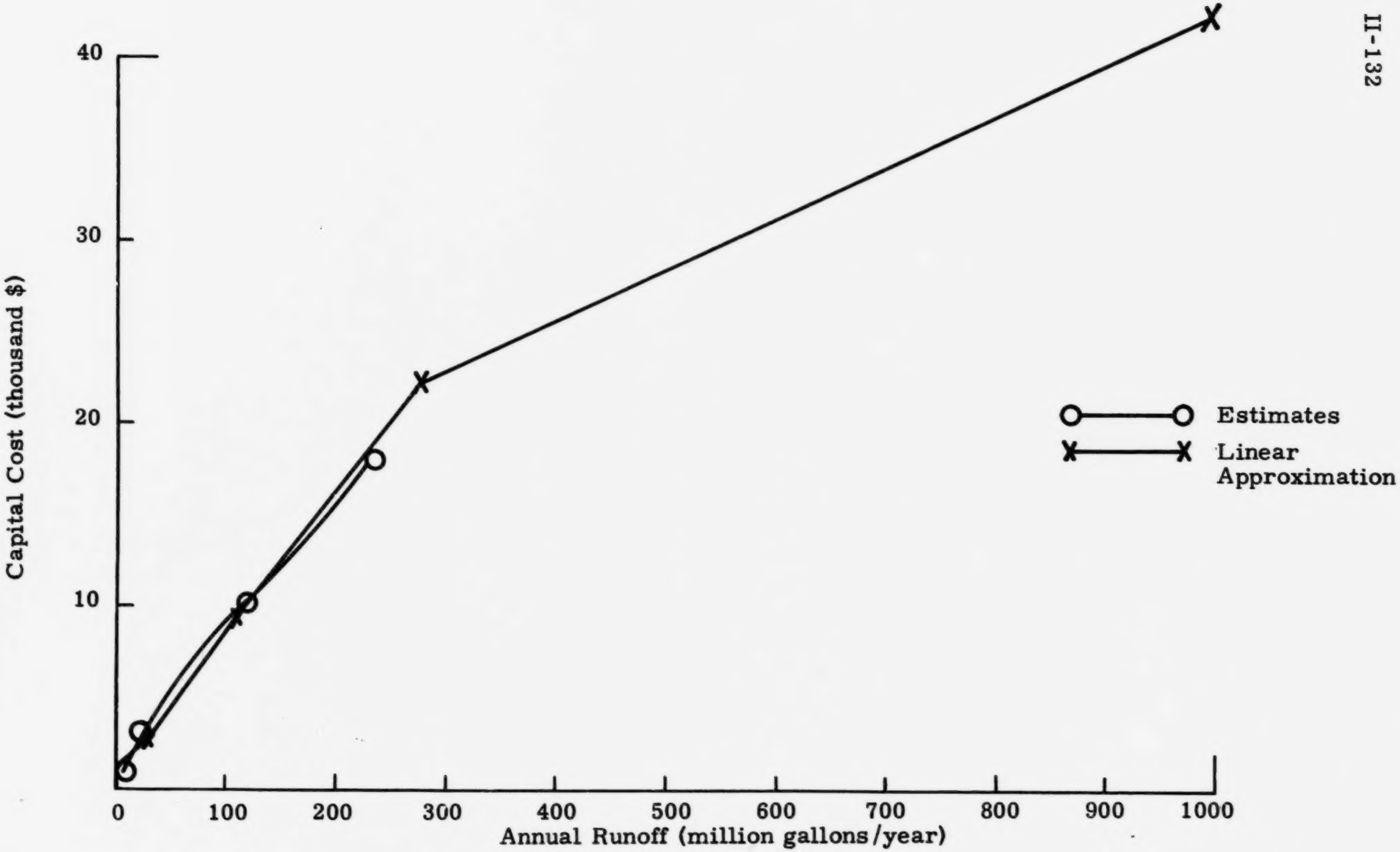


Figure II-53. Pretreatment Capacity vs. Capital Costs

TABLE II-37. SUMMARY OF CONSTRUCTION COST ESTIMATES - PRETREATMENT

(Costs in Dollars)

Capacity (gallons/day)	Excavation and Backfill	Structure, Pipes, Etc.	Tube Settler	Chemical Feeders	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
21,600	150	315	125	250	840	210	1,050
130,000	280	650	750	400	2,000	520	2,600
650,000	400	3,350	3,750	500	8,000	2,000	10,000
1,300,000	750	5,450	7,500	500	14,200	3,550	17,750

TABLE II-38. CONSTRUCTION COST VERSUS CAPACITY - PRETREATMENT

Pretreatment Capacity (overflow rate- gallons/day)	Capital Cost (dollars)
15,000	1,050
60,000	1,680
150,000	2,800
600,000	9,400
1,500,000	22,200
5,500,000	42,400

b. Final Treatment. Capital costs for the final treatment phase were developed in the same manner as pretreatment costs. Facilities for each treatment level were estimated for several capacities within the range of those being investigated. A capital cost/capacity curve was approximated and this curve, in turn, approximated by a series of straight lines. The inflection points of this last curve were reduced to a table for input to the computer program, permitting calculation of a cost for any size plant by a simple linear interpolation between the appropriate points on the table. The basic cost data are summarized on Table II-39 and Figure II-54 shows the developed curves for each of treatment levels. Since no final treatment plant is required for Class "C" water, costs are shown for Classes "B", "A", and "AA" only. Table II-40 shows the final costs input to the computer program.

TABLE II-39. SUMMARY OF CONSTRUCTION COST ESTIMATES - FINAL TREATMENT

(Cost in Dollars)

TREATMENT

CLASS "B"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Chemical Feeders	Strainer	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100, 000	500	3, 500	400	12, 400	16, 800	4, 200	21, 000
500, 000	1, 000	4, 800	500	22, 500	28, 800	7, 200	36, 000
1, 000, 000	1, 400	6, 000	650	31, 950	40, 000	10, 000	50, 000

CLASS "A"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Chemical Feeders	Filter	Monitors	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100, 000	5, 000	9, 800	400	3, 800	1, 000	20, 000	5, 000	25, 000
500, 000	14, 000	38, 000	500	12, 000	1, 000	66, 000	16, 500	82, 000
1, 000, 000	22, 000	89, 350	650	27, 000	1, 000	140, 000	35, 000	175, 000

CLASS "AA"

Capacity gal/day	Excavation and Backfill	Structure, Elec., Piping, Etc.	Filter	Chemical Feeders	Auto. Monitors	Activated Carbon Unit	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
100, 000	8, 000	15, 500	3, 800	400	5, 300	11, 000	44, 000	11, 000	55, 000
500, 000	23, 000	61, 000	12, 000	500	6, 500	45, 000	148, 000	37, 000	185, 000
1, 000, 000	35, 500	125, 000	27, 000	650	6, 500	65, 000	260, 000	65, 000	325, 000

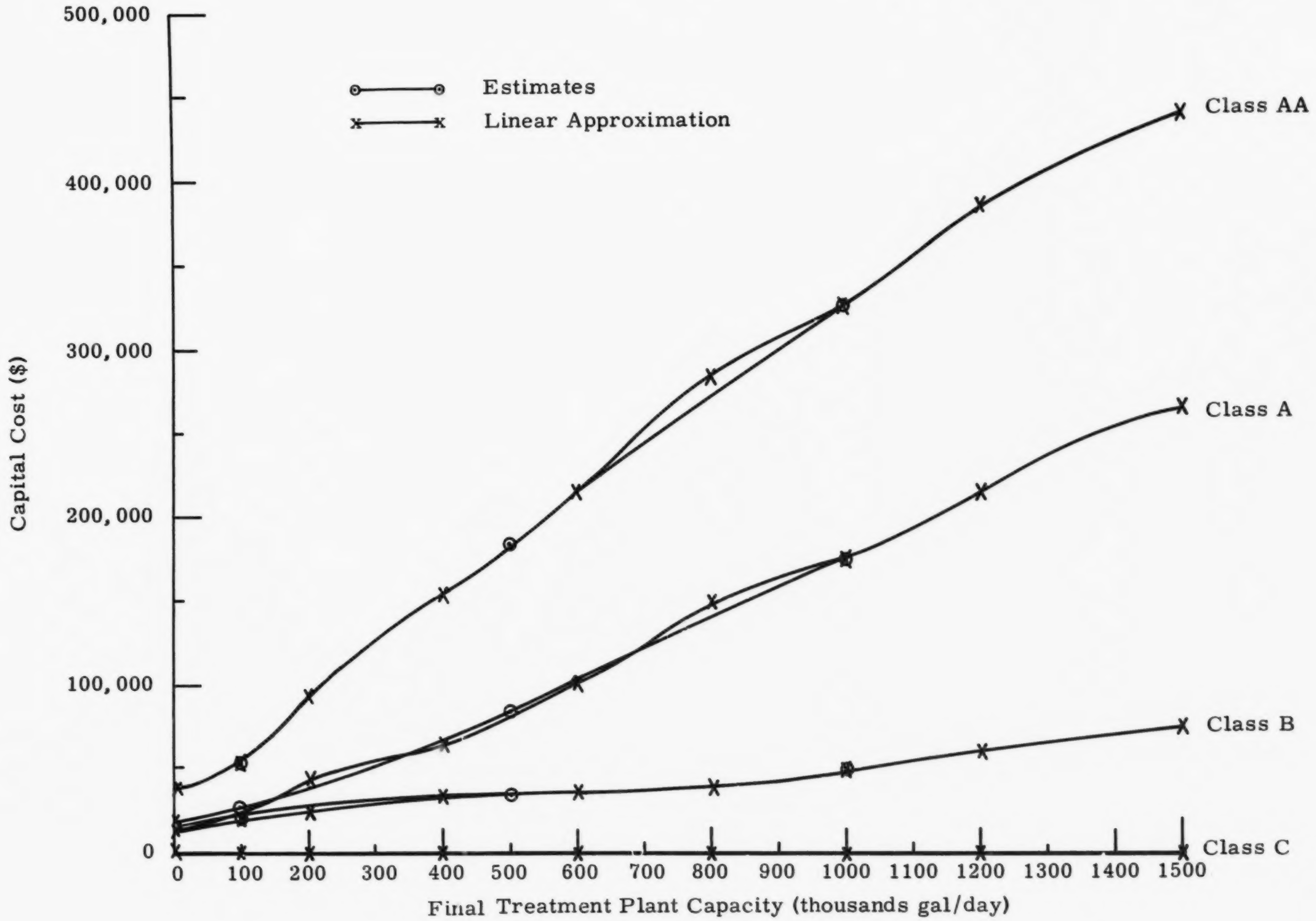


Figure II-54. Final Treatment Capacity vs. Capital Cost

As in the case of the pretreatment unit, underground construction and solution feed of all chemicals are assumed. Softening and secondary coagulation are not included in these costs. Chlorine costs are not shown separately, based on the assumption that the facility installed in the pretreatment unit would provide chlorine to both injection points.

TABLE II-40. CONSTRUCTION COST VERSUS CAPACITY - FINAL TREATMENT

	Final Treatment Capacity (gal/day)	Capital Cost (dollars)
CLASS "B"	0.0	18,000
	100,000	21,000
	200,000	25,000
	400,000	34,000
	600,000	37,500
	800,000	40,000
	1,000,000	50,000
	1,200,000	60,000
	1,500,000	75,000
CLASS "A"	0.0	15,000
	100,000	25,000
	200,000	45,000
	400,000	65,000
	600,000	100,000
	800,000	150,000
	1,000,000	175,000
	1,200,000	215,000
	1,500,000	265,000
CLASS "AA"	0.0	40,000
	100,000	55,000
	200,000	95,000
	400,000	155,000
	600,000	215,000
	800,000	285,000
	1,000,000	325,000
	1,200,000	385,000
	1,500,000	440,000

Both Classes "A" and "AA" treatment facilities will include continuous effluent turbidity monitors to insure proper operation of the filters. This monitor will operate recorders, producing a permanent record of plant operation as well as alarm systems which will be arranged to shut down the plant in the event of malfunction. It is anticipated that the maximum effluent turbidity can be maintained under 1.0 Jackson Turbidity Unit by the use of this control system.

Costs for Class "AA" treatment include more generous allowances for equipment and instrumentation. It is anticipated that product water from this type of installation would be introduced into an existing potable water distribution system, thus imposing a higher standard of reliability and fail-safe operation.

c. Pumping. Costs were developed for the high-lift pumping requirement only. The below-grade installation of the treatment units permits gravity feed between processes and the cost of backwash or flushing pumps has been included in the cost of the related piece of equipment. Pump costs include control and switchgear costs and are based on a single pump for each application. Redundant facilities were not considered due to the supplemental and decentralized nature of the facilities under study. Similarly, it was not considered appropriate to cost premium-quality, continuous-duty equipment. Table II-41 summarizes the pump cost estimates and Figure II-55 shows the development of the final cost table. This final table is shown as Table II-42.

TABLE II-41. SUMMARY OF CONSTRUCTION COST ESTIMATES - PUMPING FACILITIES

(Cost in Dollars)

Capacity (gal/day)	Pump & Motor	Controls Switchgear & Wiring	Valves & Appurt.	Const. Cost	Engr., Cont., Etc. (@25%)	Total Cost
1,000	110	30	20	160	40	200
28,800	315	50	35	400	100	500
72,000	570	300	250	1,120	280	1,400
144,000	1,220	400	300	1,920	480	2,400
720,000	4,940	700	600	6,240	1,560	7,800
1,440,000	10,000	1,400	1,000	12,400	3,100	15,500

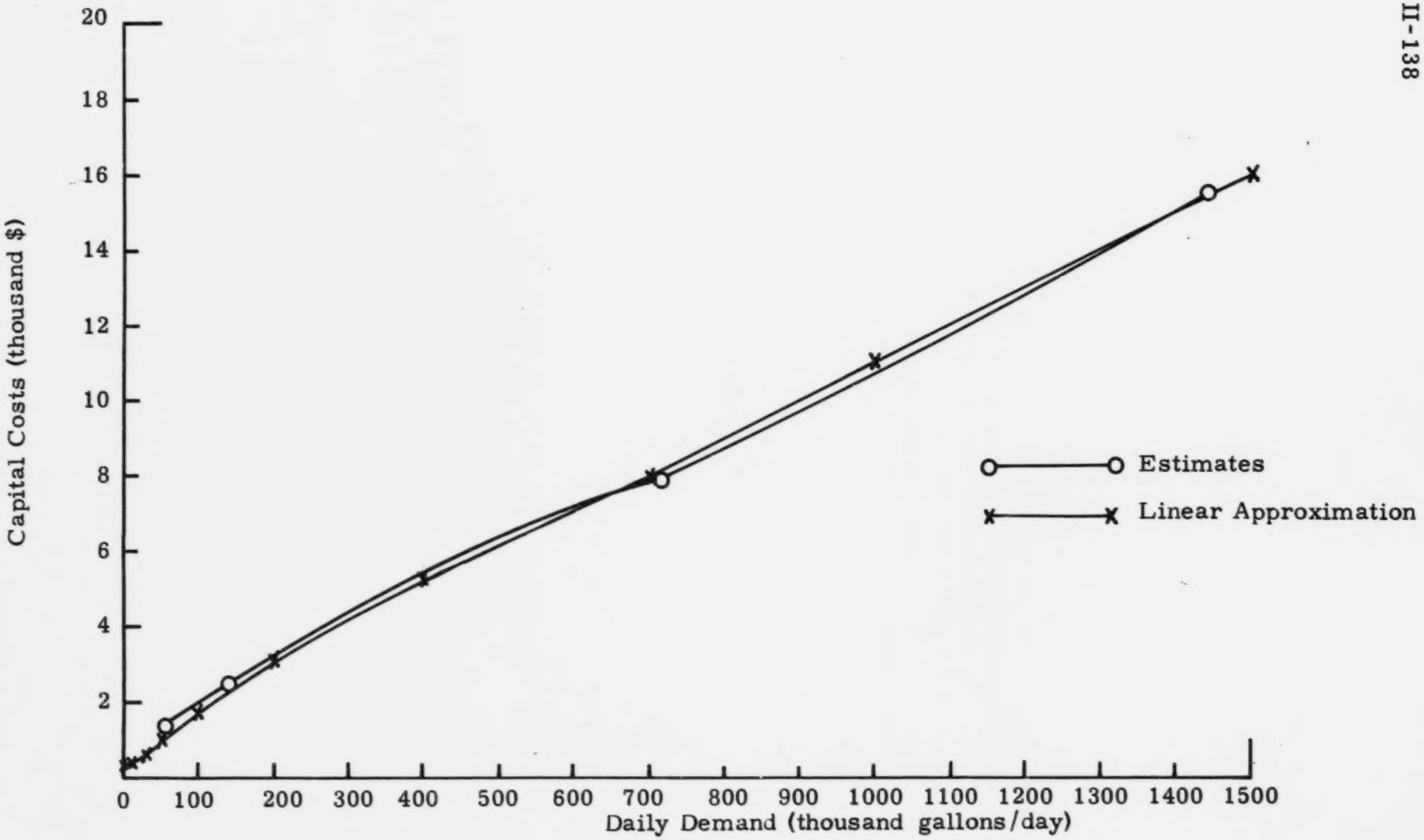


Figure II-55. Daily Demand vs. Capital Cost of Pumping Facilities

TABLE II-42. CONSTRUCTION COST VERSUS CAPACITY - PUMPING FACILITIES

Pumping Capacity (gal/day)	Capital Cost (dollars)
0.0	200
1,000	200
10,000	250
30,000	600
50,000	1,000
100,000	1,840
200,000	3,100
400,000	5,300
700,000	8,000
1,000,000	11,000
1,500,000	16,000

d. Treated Water Storage. Seven different configurations of treated water storage were considered and costs developed for each of them. These alternatives can be divided into three different modes of operation, namely:

(1) Ground level storage with direct pumping into distribution system, using storage as suction well

(2) Elevated storage supplied by pumping, storage floating on the distribution system

(3) Hydropnuematic tank floating on system and supplied by pumping

The seven storage configurations related to these modes are:

(1) Ground level - steel tank below grade

(2) Ground level - concrete tank below grade

(3) Ground level - steel tank above grade

(4) Ground level - concrete tank above grade

(5) Elevated - elevated tank on legs or pedestal

(6) Elevated - standpipe

(7) Hydropnuematic - steel tank partly below grade

The development of construction costs for all except the last configuration was discussed in Section II.F.4. Hydropnuematic tanks were treated as steel tanks constructed above grade and sized at four times the required storage. This approach simplified programming requirements and proved to be relatively accurate.

Treated water storage facilities were sized as a function of the total daily treated water demand. A mass diagram of hourly water demands was analyzed to determine storage requirements. In each case, the storage requirement was taken as 0.35 times the total daily demand. This can also be stated as an eight-hour storage requirement. Due to the various configurations considered, the actual tank size should be larger than the storage requirement. The following sizing relationships were used to obtain tank sizes:

For ground level storage and elevated storage by elevated tank -

$$\mathrm{CAPT} = 0.35\,(\mathrm{DEMC}) \tag{II-35}$$

For elevated storage by standpipe

$$\mathrm{CAPT} = 1.05\,(\mathrm{DEMC}) \tag{II-36}$$

For hydropnuematic tanks

$$\mathrm{CAPT} = 1.40\,(\mathrm{DEMC}) \tag{II-37}$$

where CAPT = tank size in gallons

DEMC = daily treated water demand in gallons

4. Operating and Maintenance Costs

Operation and maintenance costs were computed for each element of the system to include the time and material necessary to insure continued reliable operation. System operation at all levels of scale was treated as an incremental addition to a larger system. This assumes that the necessary labor, equipment, and materials would be provided or procured by an existing organization, whose overall size and organizational structure would not be significantly changed by the largest system under consideration in this study. Depending on the nature of the final system, the operation responsibility might devolve upon the Columbia Park and Recreation Association, a pseudo-governmental body which holds title to the public space in the Columbia area, including Wilde Lake and its tributaries, or to the Howard County Metropolitan Commission, a public water and sewer authority. In either case, the assumption of a small incremental addition to an existing organization is valid.

Using this approach, labor costs are taken as direct cost plus an allowance for labor fringe benefits. Materials and supplies are costed at procurement cost and equipment at established local rental rates. No provision is made for general or administrative overhead, for billing and collection, or other non-direct items, since it was considered that the addition of the system under study would not appreciably change any of these costs. Wage rates and material costs are estimated on the basis of 1967-1968 prices.

Parametric costs were developed in the same manner as the construction costs, extrapolating a curve from several calculated points. The curves were reduced to tables of points for linear interpolation by the computer program. All operating and maintenance costs are expressed in dollars per day, based on yearly averages of flows and demands.

a. Pretreatment. The principal costs associated with the operation of the pretreatment unit are the replenishment of the various chemicals employed and the periodic removal of accumulated sediment from the sump. Typical chemical requirements for this process are aluminum sulphate (alum) and calcium hypochlorite. Dosage rates have been estimated at 10 mg/1. $Al_2(SO_4)_3$ and 0.35 mg./1. available chlorine. This requires feed rates of 85 pounds of alum and 4.25 pounds of calcium hypochlorite per million gallons of runoff. Other chemicals, such as lime, polyelectrolytes, soda ash, etc., might be required to adequately treat waters in certain locations, but published storm water analyses indicate that the chemicals named will be satisfactory in most applications. All chemicals will be batch-mixed in solution tanks and fed by metering pumps. The feeders will be checked daily and chemicals replenished as required. Computation of operating costs did not include removal of sediment from the pretreatment unit sump, as this operation was combined with cleaning and inspection of the storm water reservoir and described with parametric costs in Section II. F.5. Chemical costs, electric power costs, and costs associated with regular attention to proper operation are calculated for the pretreatment unit and shown in Tables II-43 and II-44, as well as in Figure II-56. These costs can be identified as operation and maintenance costs for Class "C" treatment and are included in the costs shown for higher levels of treatment.

b. Final Treatment. Operation of the final treatment facilities consists of routine inspection of the various items of equipment, daily tests of water quality to verify chemical feed rates and to check automatic monitors, backwashing of filters or strainers, replenishment of chemical feeders, etc. Preventive maintenance is carried out on the various pieces of equipment and minor replacement of components is performed as indicated. Solution-type chemical feeders are used to add a precoat material to the straining operation for Class "B" treatment and to provide a chemical coagulant where required to the Class "A" and Class "AA" plants. It was assumed for costing purposes that this coagulant would be aluminum sulphate (alum), although in practice, other materials might be employed and the addition of alkalinity and pH adjusting chemicals, such as lime or soda ash, might be required. Rechlorination will be practiced at all treatment points, using calcium hypochlorite. Feed rates are estimated at 50 pounds $Al_2(SO_4)_3$ and 8.5 pounds $Ca(OCl)_2 \cdot 4H_2O$ per million gallons of treated water. Daily attention to all treatment equipment has been assumed.

Estimates also include energy cost for electric power to mixers, metering pumps, sump pumps, etc., as well as power for high service pumping.

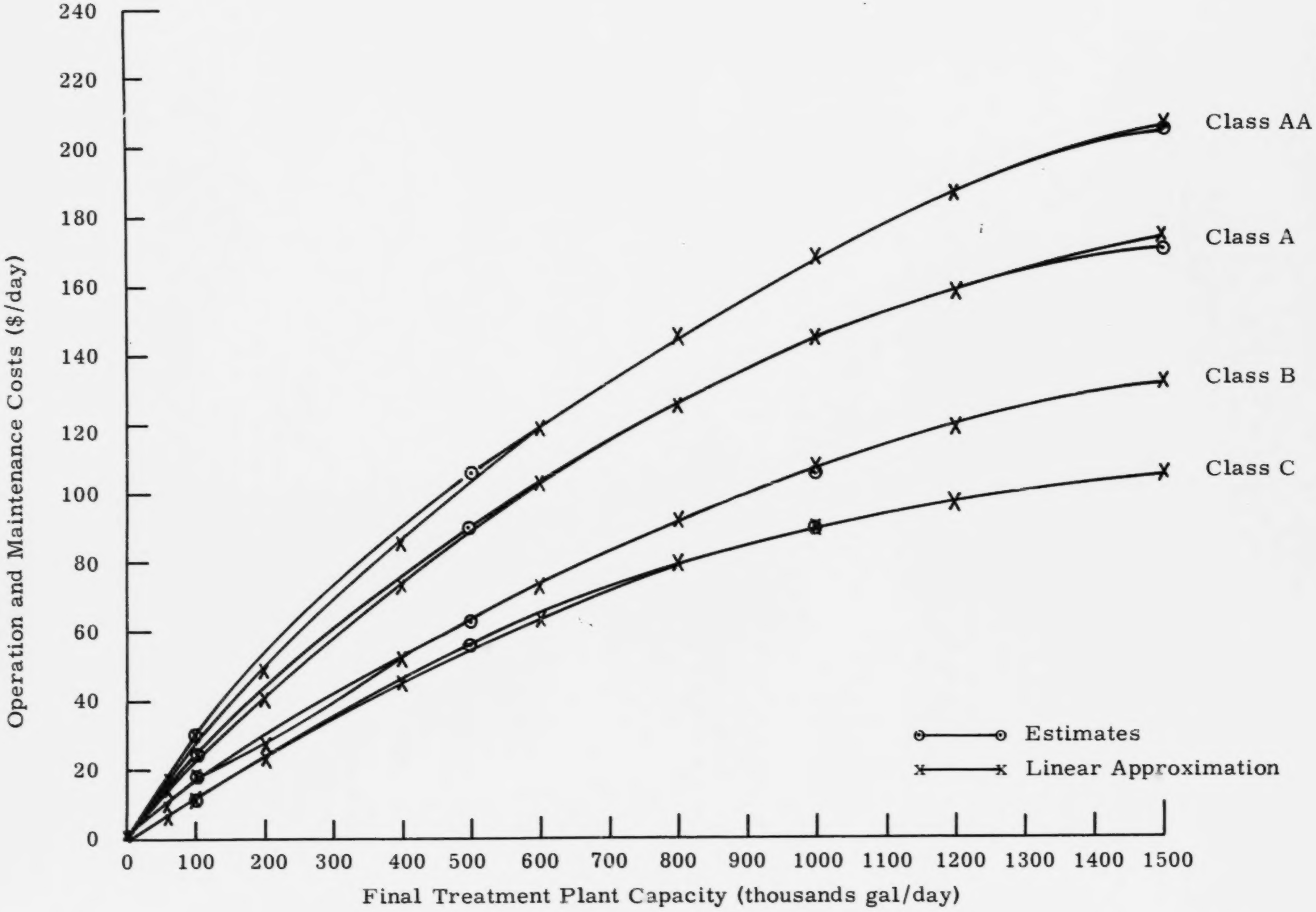


Figure II-56. Operation and Maintenance Cost vs. Capacity - Final Treatment

TABLE II-43. SUMMARY OF OPERATING COST ESTIMATES - TREATMENT

	Capacity Gal/Day	Operation		Chemical		Electric Power $/Day	Misc. Main. Supplies $/Day	Misc. Oper. Supplies $/Day	Contingency $/Day	Total O&M $/Day
		Man/Hrs	Rate	Cost	$/Day					
CLASS "C"	100,000	0.65	3.50	2.25	0.45	8.50	0.50	0.20	0.50	12.00
	500,000	2.00	3.50	7.00	2.25	40.50	4.25	2.00	2.50	55.00
	1,000,000	5.00	3.50	17.50	4.50	81.00	35.00	25.00	5.00	168.00
CLASS "B"	100,000	1.50	3.50	5.25	0.75	9.00	1.00	1.00	1.00	18.00
	500,000	3.50	3.50	12.25	3.75	42.50	1.50	1.50	1.50	63.00
	1,000,000	3.50	3.50	12.25	4.25	84.00	1.50	1.50	1.50	105.00
CLASS "A"	100,000	3.00	3.50	10.50	0.85	9.00	1.15	1.50	1.50	25.00
	500,000	10.00	3.50	35.00	4.25	42.50	2.75	3.50	2.00	90.00
	1,000,000	13.00	3.50	45.50	8.50	85.00	3.50	4.50	3.00	150.00
CLASS "AA"	100,000	3.50	3.50	12.25	1.50	10.00	1.50	1.75	3.00	30.00
	500,000	11.00	3.50	38.50	7.00	43.50	5.00	6.00	4.00	105.00.
	1,000,000	14.00	3.50	49.00	14.00	86.00	20.00	31.00	5.00	205.00

Table II-43 lists the detailed costs used to develop operating and maintenance costs for various sizes of treatment facilities and Figure II-56 shows the extrapolated cost curve, as well as the approximated curve. The table of values input to the computer program is shown as Table II-44.

TABLE II-44. OPERATION AND MAINTENANCE COST VERSUS CAPACITY - TREATMENT

Treatment Capacity (gal/day)	Operating & Maintenance Cost ($/day)			
	Class "C"	Class "B"	Class "A"	Class "AA"
0.0	0.00	0.00	0.00	0.00
60,000	7.00	9.00	15.00	18.00
100,000	12.00	19.00	24.00	28.00
200,000	24.00	27.00	41.00	48.00
400,000	46.00	52.00	74.00	86.00
600,000	64.00	73.00	103.00	119.00
800,000	80.00	92.00	126.00	146.00
1,000,000	90.00	108.00	145.00	168.00
1,200,000	97.00	119.00	159.00	186.00
1,500,000	106.00	132.00	174.00	206.00

c. Pumping. Operating and maintenance cost of the high service pumping facilities have been included in the Final Treatment costs detailed in Section II.F.4.b.

d. Treated Water Storage. Operation and maintenance costs were computed separately for each of the seven storage options described in Section II.G.3.d. These costs consist of periodic painting and repair of the tanks and are a function of the surface area to be maintained. Section II.F.5 describes the methods employed to calculate the surface area for various types of cylindrical tanks. Elevated tanks may be cylindrical, but are more frequently ellipsoidal. In addition, they stand on a pedestal or on legs which have appreciable surface area themselves. A variety of tank shapes and sizes were analyzed and the following equations approximate the surface areas of most elevated tanks:

$$\text{ASTO} = 6.283185\,(\text{AA})^2 + \frac{3.14159\,(\text{BB})^2}{\text{EE} \times \ln\frac{(1 + \text{EE})}{(1 - \text{EE})}} \qquad \text{(II-38)}$$

where: ASTO = surface area of one side of ellipsoidal tank (sq. ft.)
AA = $\frac{1}{2}$ tank daimeter (found by using Equation II-31) (ft.)
BB = $\frac{1}{2}$ tank height (found by using Equation II-31) (ft.)
EE = $(AA)^2 - (BB)^2 \ \frac{1}{2}/AA$
ln = natural logarithm

$$AST3 = ASTO - (200\ CAPT/10^6)^{\frac{1}{2}} \qquad \text{(II-39)}$$

where: AST3 = total outside surface area, including support (sq. ft.)
CAPT = tank capacity (gallons)

In calculating total maintenance cost for an elevated tank, ASTO is taken as the inside surface area and AST3 as the outside area. In the present analysis, both cases are multiplied by $0.05 per square foot per year and the sum of the products is the yearly painting and maintenance cost. The cost factor is described further in Section II. F. 5.

H. Reuse Distribution Systems

1. Distribution Systems and Reuse Class

a. General Criteria. The distribution system for storm water reuse is comprised of all the pipes, valves, meters, and appurtenances required to conduct the water from the point of treated water storage to the point of ultimate use. It includes connections to the existing public water system for makeup water with anti-backflow provisions at the interface between the two systems.

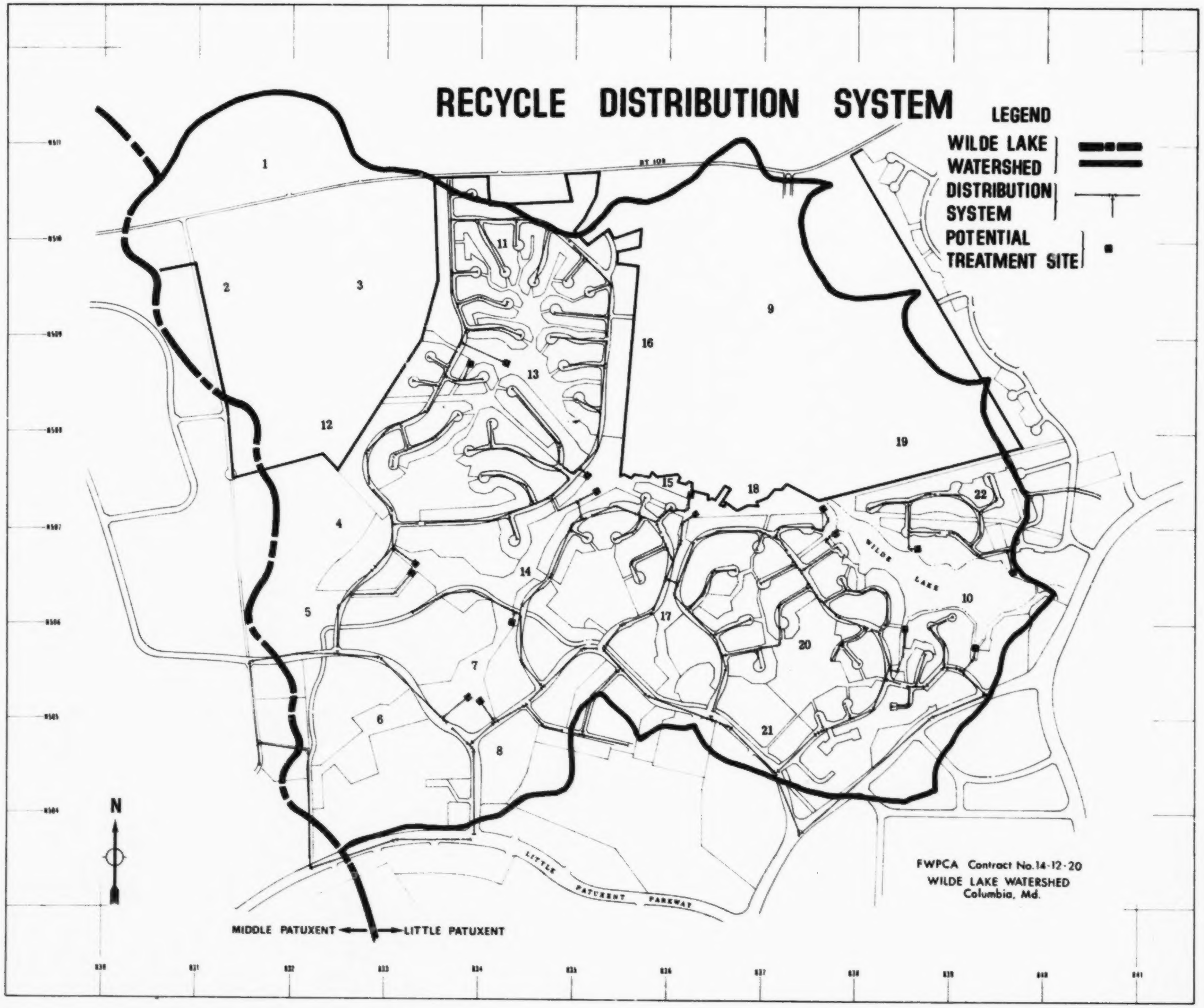
Pipe sizes throughout the distribution system have been selected for the estimated peak flows as well as reasonable ease of maintenance. Fire flows have not been assumed, since the public system will retain that function, consistent with the discussion in Section II.D. No pipes smaller than 2" have been assumed and the largest pipe diameters are 6". Where a sub-potable distribution system serves a property, double meter vaults have been assumed with meters on both potable and sub-potable systems. Costs for sub-potable distribution systems within buildings include the additional costs of dual risers, extra fittings, etc., but do not include fixture costs. Sub-potable distribution systems have been designed for each of the distribution service areas described in Section II.D and laid out in such a way that any combination of continuous systems can be interconnected by simply operating valves (see Figure II-57). This permits the evaluation of systems for various combinations of sub-watersheds without changing distribution costs, as well as providing for continued operation in the event of temporary failure of one system.

The transmission main from the treatment plant to the distribution piping has been calculated as a function of plant location only. The cost includes the public water system connection, which includes provisions for providing makeup water to sub-potable systems without the possibility of backflow into the public system. This might be accomplished by introducing public water into an open tank through a float-controlled valve located above the overflow, or, local regulations permitting, by one of several types of backflow preventer valves. In the case of potable reuse systems, the public connection is the point of injection into the public potable water distribution system. Some type of backflow preventing device might be employed here to isolate section of the distribution system, depending on local conditions.

b. Class "C" and "B&C" Uses. When treated storm water is to be used to flush toilets only or to sprinkle lawns and flush toilets, a relatively simple dual distribution system is required. In addition to a sub-potable distribution piping in the streets, dual service connections and meters are required at each home, along with dual headers on the individual plumbing. Connections from the sub-potable system are made to the outside hose bibbs for sprinkling use and to toilet fixtures throughout the building.

c. Class "B" Uses. Reuse of treated storm water exclusively for lawn sprinkling requires a sub-potable distribution system and dual service connections, but minimizes the cost of plumbing alterations in each building. The sub-potable system is confined to the basement or ground level and is connected to outside hose bibbs only.

d. Class "A", "A&B", "A&C", and "A, B, &C" Uses. These use combinations also require a sub-potable distribution system with dual


RECYCLE DISTRIBUTION SYSTEM
LEGEND
WILDE LAKE WATERSHED
DISTRIBUTION SYSTEM
POTENTIAL TREATMENT SITE ▪
RT 108
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
WILDE LAKE
LITTLE PATUXENT PARKWAY
N
MIDDLE PATUXENT
LITTLE PATUXENT
FWPCA Contract No.14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
8511
8510
8509
8508
8507
8506
8505
8504
830
831
832
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841

Figure II-57. Distribution System Map - Wilde Lake Watershed

service connections to each building. The sub-potable plumbing within the building, however, must be provided to each plumbing fixture except kitchen and bathroom sinks. In many homes, and particularly in apartment buildings, this will require virtual duplication of the entire cold water system. It is assumed that the hot water system could be split in multi-family units and perhaps in single family residences.

e. Class "AA, A, B, & C" Uses. When storm water is treated to potable water quality, the distribution system is greatly simplified. The transmission line is provided from the treatment plant to the existing public water supply main where a metered connection is provided. The existing public system is then permitted to distribute the potable water to all water uses within the service area. It was anticipated that the service areas might be divided by check valves or division valves, but in practice this would probably not be desirable and the output of the treatment plant would supplement the supply throughout the system.

2. Construction and Operating Costs

a. Construction Costs. All costs for the distribution systems were estimated on the assumption that the necessary piping and plumbing would be installed following construction of the streets and buildings, and that no particular economies would be realized from concurrent construction with conventional facilities. Underground pipelines have been assumed to be cast iron and buried with a minimum of four feet of cover, with conventional valves and fittings. Individual building connections and plumbing alterations have been priced on a per unit basis. The in-place cost of cast iron pipe was taken at $7.50 per foot of 6" pipe and $4.75 per foot of 2" pipe. Table II-45 lists the estimating figures used for building service connections and for interior plumbing. All figures include materials, labor, overhead, engineering and construction contingencies and are 1967-68 prices. These prices are applicable to sub-potable distribution systems only and include provision for identification of pipes and valves by color coding, tagging, etc.

b. Operation and Maintenance Costs. Operation and maintenance of sub-potable distribution systems is primarily composed of occasional repair of distribution mains, the portion of the service connections in the public right-of-way, and routine inspection and reading of the individual meters. Only the incremental cost of reading the additional meter was computed, since it would be read simultaneously with the existing potable water meter. These costs were estimated on a yearly basis as 0.006 times the construction cost of the system. The formula employed to convert these costs to daily costs is:

$$\text{DACOS} = \text{ATOT}\ (0.006/365) \qquad \text{(II-40)}$$

where:

ATOT = Total construction cost of distribution system

DACOS = Daily operation and maintenance cost of system

The portion of the service connection which is located on private property and the internal plumbing would be maintained by the individual property owner in the same manner as the existing conventional plumbing which it supplements.

TABLE II-45. CONSTRUCTION COSTS FOR SERVICE CONNECTIONS AND INTERNAL PLUMBING FOR SUB-POTABLE REUSE

Type of Occupancy	"C", "B&C"	"B"	"A", "A&B", "A&C", "A, B, &C"
Single family residential, medium density	$300/unit	$200/unit	$340/unit
Single family residential, low density	$300/unit	$200/unit	$340/unit
Town houses	$300/unit	$200/unit	$340/unit
Garden apartments	$1000/20 units or fraction plus $65/unit	$1000/20 units or fraction	$1000/20 units or fraction plus $100/unit
Mid-rise apartments	$ 80/unit	------	$100/unit
Schools, 500 pupils	$1000/building	$1000/building	$1000/building
Commercial uses	individually estimated		

3. Input to System Model

The construction cost for each service area, or sub-watershed, was computed for each group of reuse classes as described in the preceding paragraphs. These values were used as input to the computer program, permitting the selection of the proper costs for and sub-watershed or, by adding costs for component areas, the proper value for any combination of sub-watersheds. The costs of transmission lines from treatment plants were computed separately for each treatment plant location. These values are shown as Tables II-46 and II-47, respectively.

TABLE II-46. CONSTRUCTION COST VS. SERVICE AREA - DISTRIBUTION SYSTEM

Sub-Watershed	Construction Cost for Reuse Level (Dollars) "A"	"B"	"C"	"A", "B"	"A", "C"	"A", "B", "C"	"B", "C"
1	17,540	14,600	16,700	17,540	17,540	17,540	16,700
2	20,990	18,050	20,150	20,990	20,990	20,990	20,150
3	10,330	9,210	10,010	10,330	10,330	10,330	10,010
4	39,060	27,300	35,700	39,060	39,060	39,060	35,700
5	45,800	30,120	41,320	45,800	45,800	45,800	41,320
6	269,920	163,400	238,930	269,920	269,920	269,920	238,930
7	141,760	85,900	125,520	141,760	141,760	141,760	125,520
8	54,500	31,900	47,720	54,500	54,500	54,500	47,720
9	151,300	136,600	147,100	151,300	151,300	151,300	147,100
10	135,650	92,250	122,575	135,650	135,650	135,650	122,575
11	80,890	63,950	76,050	80,890	80,890	80,890	76,050
12	44,180	36,200	41,900	44,180	44,180	44,180	41,900
13	54,450	41,850	50,850	54,450	54,450	54,450	50,850
14	110,820	82,000	102,300	110,820	110,820	110,820	102,300
15	22,460	18,120	21,220	22,460	22,460	22,460	21,220
16	82,760	70,300	79,200	82,760	82,760	82,760	79,200
17	108,720	78,660	104,680	108,720	108,720	108,720	104,680
18	31,320	24,840	29,440	31,320	31,320	31,320	29,440
19	56,360	50,200	54,600	56,360	56,360	56,360	54,600
20	101,240	78,840	93,440	101,240	101,240	101,240	93,440
21	83,330	50,910	72,415	83,330	83,330	83,330	72,415
22	33,860	26,020	31,620	33,860	33,860	33,860	3,162

TABLE II-47. CONSTRUCTION COSTS VS. SERVICE AREA-TRANSMISSION LINES

Treatment Plant Location (Sub-watershed)	Construction Cost (Dollars)
1	3000
2	3000
3	4500
4	2250
5	5550
6	2630
7	750
8	1800
9	2250
10	1950
11	3900
12	1500
13	750
14	1500
15	2400
16	3750
17	4280
18	2850
19	1730
20	1580
21	4280
22	1500

III. SYSTEM OPTIMIZATION

A. Introduction

The system study of the local storage, treatment and reuse of storm water concept has been outlined in the Introduction to Chapter II. The system was defined and illustrated schematically in Figure II-1 and II-2. The objective function was stated as:

(1) Water released to surface drainage from the Local Storage, Treatment, and Reuse of Storm Water system shall meet or exceed stated effluent standards.

(2) This condition will be achieved at the lowest net system cost, or maximum net system benefit.

Maximum net system benefit was, in turn, defined as:

$$\max (\Sigma B_a - \Sigma C_a - a\Sigma C_f)$$

Where: B_a = marginal annual benefits

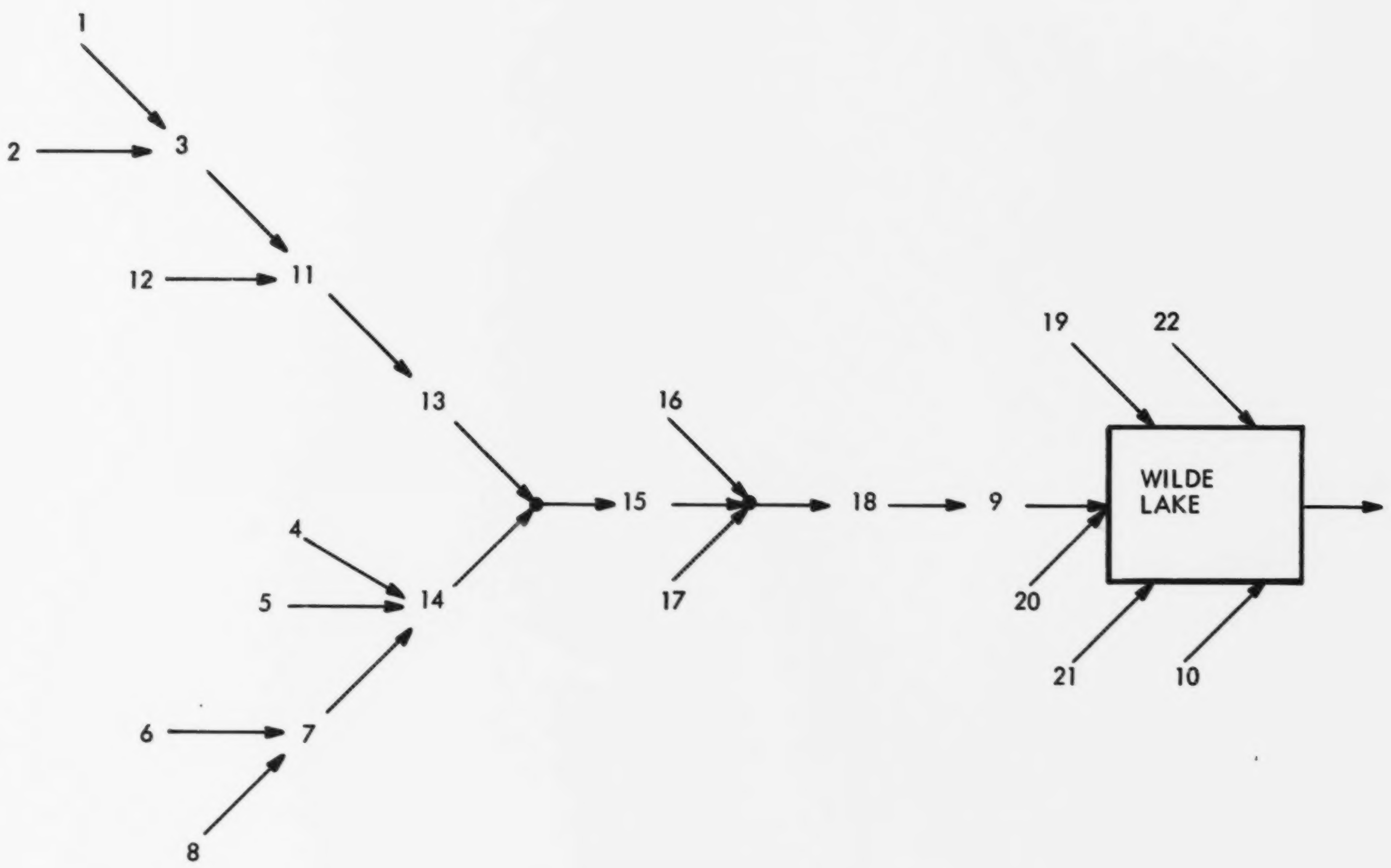
C_a = marginal annual operating and maintenance costs

C_f = marginal fixed construction and project costs

a = capital recovery factor (See page II-4)

The effluent standards required by the first part of the objective function were developed in Section II-E and shown as Table II-11. The constraints on the objective function were identified and investigated and the results reported in various sections of Chapter II. All information obtained concerning these constraints was prepared in a suitable form for input to a digital computer code for maximization of the objective function subject to the constraints.

A review of the studies reported in Chapter II indicates a large number of possible physical systems, all of which would satisfy the stated physical, technological, and institutional constraints. Twenty-two sub-watersheds were considered, and a local system could be placed in each of them. In this manner, the total watershed could be served by twenty-two local reuse systems, making up the overall system for the watershed. This is only one of many ways to meet the requirements of the watershed, however. There are many points in the watershed where two, three, or more sub-watersheds can be collected at one point. Figure III-1 shows the flow pattern of the various sub-watersheds and can be used to determine feasible combinations of multiple sub-watersheds. In these cases, the combined sub-watersheds are treated as a single area, and have a single local reuse facility. As many as eighteen sub-watersheds can be combined into a single collection area, and four which discharge directly into Wilde Lake must remain independent. An exhaustive analysis of the flow diagram would reveal that more than 10,000 combinations of various sized collection areas are possible. Fortunately, many of these are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WILDE LAKE
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10

Figure III-1. Wilde Lake Drainage Area Sub-Watershed Flow Pattern

trivial, and others present varying degrees of difficulty in terms of physically intercepting the storm flows at the required points. Close examination of the watershed resulted in 173 workable combinations of service areas, each comprised of some number of sub-watersheds, ranging from one to six. In view of the study objective of evaluating local storage, treatment, and reuse of storm water, an upper limit of six sub-watersheds, or about 350 acres, was set for the initial attempt at system optimization. It was planned to reconsider this decision after the first trial, but the results, reported below, indicated that the optimum had been located within the range selected.

Each of the 173 overall systems is made up of a number of sub-systems, namely, the various storage units, pretreatment units, final treatment units, treated water storage units, distribution systems, etc., that comprise the facilities in each collection area. The storm water might be treated to any of four qualities and the distribution system might be constructed to connect to any of eight different combinations of water uses. Within a given overall system, the individual facilities might vary from each other. Each of these possible sub-systems and overall systems must have construction, operating, and maintenance costs calculated, adjusted to annual costs, and subtracted from the net benefit that would be derived from the overall system. When the remainder from this calculation is available from each of the possible systems, the optimum system can be identified and described. Due to the great complexity and magnitude of the calculations required, the system optimization was performed by a digital computer. The following sections describe the preparation of inputs to the computer program, the operation of the program, and the type of outputs obtained. The system optimization results will be described, and final investigation of alternatives reported.

B. System Model Inputs

1. Design Criteria

a. Storage Facilities. In order to insure that the systems under analysis all correspond to the range of technologic and physical possibilities, the essential design criteria have been determined and must be included as inputs to the system model, prior to optimization wherever possible. In the case of storm water storage, various types of storage were considered, which might have varying applicability in different locations. Steel and concrete tanks, both buried and at grade, were considered, along with open ponds. The system model was arranged for a sub-optimization of storage facilities, based on economic factors, which is described later. In the event that local conditions prohibit a free choice among the alternatives, the program can be instructed to choose either open ponds, buried tanks, or non-buried tanks as the only possible construction method. It can also be instructed to choose only steel tanks or only concrete tanks, when tanks are being considered.

Section II-F described the analysis performed to develop sizing criteria for reservoirs. It was determined that the various reservoir purposes could be served adequately by sizing the storm water storage facility at the maximum expected volume of a storm just meeting the one-year return interval criteria. The one year storm volume has been calculated for each sub-watershed, using the results of the hydrological analysis reported in Section II-C, and these values are listed as Table III-1.

Table III-1. Expected Maximum Volume of One-Year Storms

Sub-watershed	One-year Storm Volume (gal)
1	460,800
2	586,500
3	530,000
4	341,600
5	530,500
6	1,104,000
7	803,200
8	625,900
9	1,460,000
10	1,258,000
11	854,000
12	594,200
13	696,800
14	1,030,000
15	236,400
16	782,000
17	1,130,000
18	290,000
19	923,500
20	1,203,000
21	690,000
22	358,400

Since it was originally anticipated that storage volumes would be a function of demand, the systems analysis computer program was arranged to test the service area demand against the available supply before computing storage volume. The total runoff expected from each sub-watershed was calculated by methods described in Section II-C. Further analyses described in later section of Chapter II disclosed that, given the design criteria selected for the Wilde Lake watershed, 90 percent of the total runoff will be available for reuse at release rates of at least 1.13 $\bar{q}$. Accordingly, the total demand of any service area under consideration can be compared to 90 percent of the total runoff of that same area to determine the availability of the required volume of treated water. When the demand exceeds the available supply, it is reduced to equal that supply. This insures that later calculations of system costs, benefits, etc., refer only to the portion of the demand that will be satisfied by treated storm water. The balance of the demand will be satisfied by the public water supply system and will not figure in the calculation of costs or benefits. When the demand does not exceed the available supply, subsequent calculations will be based on the actual demand. Table III-2 lists the available supply for each of the sub-watersheds, equal to 90 percent of total runoff.

TABLE III-2. EXPECTED AVAILABLE SUPPLY FROM STORAGE

Sub-Watershed	Available Supply (gal/day)
1	8,946
2	11,565
3	10,064
4	11,807
5	19,080
6	43,513
7	33,915
8	26,468
9	43,994
10	61,997
11	30,552
12	18,090
13	24,989
14	38,340
15	9,129
16	25,327
17	50,305
18	9,630
19	38,520
20	50,193
21	28,734
22	13,410

When the service area under consideration consists of more than one sub-watershed, storage volume and maximum demand can be found by adding the values associated with each of the component sub-watersheds on the tables above.

b. Pretreatment. Operating criteria developed in Section II-G for the pretreatment unit stipulated that it must be capable of operating at twice the mean rate of runoff. This can be accomplished within the system model by inputing a table of total annual runoffs from each sub-watershed (Table III-3) and using these figures to determine costs for the pretreatment unit. Furthermore, when the total system of sub-watersheds is analyzed, each downstream pretreatment unit must process twice the mean runoff from its own watershed as well as the total of the treatment rates of all the upstream units. The cost figures used for determining construction and operating cost of pretreatment units (Tables II-37 and II-43) are based on total runoff and treated water demand, respectively, and take into account the operating rule suggested for the unit. Construction costs are calculated to provide the necessary capacity, using the total annual runoff from Table III-3.

TABLE III-3. TOTAL ANNUAL RUNOFF VS. SUB-WATERSHED

Sub-Watershed	Total Annual Runoff (gallons)
1	3,626,640
2	4,693,608
3	4,073,400
4	4,790,547
5	7,732,769
6	17,653,330
7	14,006,578
8	10,740,789
9	17,817,840
10	25,123,680
11	12,377,880
12	7,332,120
13	10,133,568
14	15,528,105
15	3,700,224
16	10,259,712
17	20,388,024
18	3,905,208
19	15,636,600
20	20,351,232
21	11,652,552
22	5,439,960

c. Final Treatment. No physical or technological constraints are placed on final treatment facilities, other than their categorization into four classes of treatment, as discussed in Chapter II. Class "C" consists of pretreatment only and only a portion of its operating costs are ascribed to final treatment. Classes "B", "A", and "AA" are composed of pretreatment and various levels of additional treatment. In all cases, demand is selected as the parameter for determining costs, and the demand is tested prior to calculation of treatment costs by the method described above in the paragraph headed "Storage Facilities."

d. Pumping Facilities. Pumping facilities are sized by adjusted treated water demand and are constrained only by cost.

e. Treated Water Storage. As in the case of storm water storage, a number of storage configurations are analyzed. Steel and concrete tanks, above or below grade, as well as elevated tanks, standpipes, and hydropneumatic tanks are all estimated for each application, unless instructions are input to do otherwise. The program can be instructed to select any of the storage configurations without reference to the others. Treated water storage tanks are sized as a function of demand by the relationships shown as Equations II-35 through II-37.

f. Distribution System. The various physical constraints on distribution systems were discussed in Section II-H and were taken into account in developing distribution system costs. Distribution costs are a function of the sub-watershed under study and the level of reuse being considered. Sub-watersheds can be combined by simply adding distribution costs. The transmission line to the treatment plant itself is listed separately (Tables II-46 and II-47) and is a function of treatment plant location only. Where one treatment plant serves two combined sub-watersheds, the two distribution costs are summed, but only the transmission line costs for the actual location of the plant apply.

2. System Costs

a. Storage Facilities. Construction, operating, and maintenance costs have been developed for five different types of storm water storage facilities and presented as tables of cost/capacity relationships. Capital costs are composed of land cost, site preparation costs, burial costs where applicable, and erection costs. Tables II-29, II-30, II-32, and II-34 list these costs. Average cost of land in the Columbia area was taken as $12,000 per acre. Later discussions with the developer of Columbia led to a decision to confine facilities to land already dedicated to public use, land not usable for other purposes due to its function as natural flood plain. Since the cost of this land was not chargeable to the local reuse system, land costs were set at $0.00 per acre for purposes of systems analysis. An auxiliary computation indicated that total system costs were reduced approximately seven percent by this assumption. Land area, for purposes of computing land cost and site preparation costs, is found from a land area table (Table II-25) in the case of open ponds, and computed from capacity and H/D ratio (Table II-26) in the case of tanks. Operation and maintenance costs are made up of sediment removal costs and metal tank maintenance. Methods for computing these costs were given in Section II-F. After evaluation of all component costs, the total system cost of a storm water storage facility of any desired configuration can be expressed as:

$$C_j = a(L_j + P_j + B_j + E_j)/365 + (R_j + M_j) \qquad \text{(III-1)}$$

Where:

C_j = total system cost of storage facility of configuration j (dollars/day);

a = capital recovery factor (See page II-4);

L_j = land cost for storage facility of configuration j (dollars);

P_j = site preparation cost for storage facility of configuration j (dollars);

B_j = burial cost, where applicable (dollars);

E_j = erection cost (dollars);

R_j = silt removal cost (dollars/day);

M_j = tank maintenance cost, where applicable (dollars/day).

All system costs were calculated as dollars/day to simplify the output format. The annual rate of interest was taken at the 1967 level recommended by the Water Resources Council for evaluation of water resources projects -- 0.032 or 3.2 percent annually. It was recognized that various persons and agencies interested in the storm water reuse concept will have a range of interest figures which they consider appropriate to the analysis. A common approach is to select a rate based on the current cost of new capital to the agency considering construction of such a facility. This might range from 4.5 percent for the Federal Treasury (1967) to approximately 5.0 percent for most local governments in late 1967 or early 1968, to perhaps as much as 7.0 percent for a private developer. Since the local reuse system is being compared to a conventional approach which is characterized by relatively high capital costs and low operating costs as compared to the study concept, the selection of a high interest rate would tend to penalize the conventional central system. Accordingly, the 3.2 percent figure was selected as the lowest interest rate that might be considered for the analysis, a conservative assumption from the standpoint of evaluating the local reuse concept. The amortization period is likewise a subject of some difference of opinion. When such considerations are influenced by financing details, the life of the bonds issued to fund the project is a common criteria -- frequently about 30 years. From an economic sense, the estimated useful life of the facilities might be used, preferably each individual unit but possibly the average useful life of all facilities. Fortunately, in this case no substantial disagreement existed between the two approaches. The estimated useful life of all facilities under consideration is considered to average approximately 30 years. For this reason, 30 years was selected as the period of amortization.

The methods employed to calculate the total system cost on a dollar/day basis of each of five possible configurations of storage have now been outlined. Following these calculations, a sub-optimization is performed for storm water storage configuration. Unless constrained to fewer choices by the options described in Section III-B-1, the five system costs are compared and the lowest total system cost is selected. Results of the other four computations are destroyed and the selected system costs are retained as the final costs of storm water storage.

b. Pretreatment. Chapter II described the development of capital costs for the pretreatment unit. Total annual runoff is obtained for a given watershed from Table III-3 and the pretreatment cost for this capacity is found by linear interpolation of Table II-37. Sediment removal costs are computed as part of the storage facility costs and other operating and maintenance costs are combined with final treatment costs. Total system cost of the pretreatment unit is defined as:

$$C_p = a(S_p)/365 \qquad \text{(III-2)}$$

Where: C_p = total system cost of pretreatment unit (dollars/day)

a = capital recovery factor (See page II-4)

S_p = construction cost of pretreatment unit (dollars).

c. Final Treatment. Final treatment costs are composed of construction and operating costs, each computed vs. daily treated water demand for four different levels of treatment. Costs for given demand and level of treatment can be obtained by linear interpolation of Tables II-39 and II-43 with respect to demand, using the sections pertaining to the treatment level selected. In this manner a construction cost and an operating cost are selected, and they are combined in the following manner:

$$C_t = a(S_t)/365 + O_t \qquad \text{(III-3)}$$

Where C_t = total system cost of final treatment (dollars/day);

a = capital recovery factor

S_t = construction cost of treatment unit (dollars); and

O_t = operating and maintenance cost of treatment unit (dollars/day).

d. Pumping Facilities. Construction costs of pumping facilities are obtained by linear interpolation of Table II-41. Operating and maintenance costs have been combined with final treatment costs. Total system costs for pumping are found by the following expression:

$$C_h = a(S_h)/365 \qquad \text{(III-4)}$$

Where C_h = total system cost of pumping (dollars/day);

a = capital recovery factor

S_h = construction cost of pumping facility (dollars).

e. Treated Water Storage. Seven different configurations of treated water storage facilities were analyzed and costs developed for each of them. The types of facilities examined include standpipes, elevated tanks, hydropnuematic tanks, ground level steel and concrete tanks both above and below grade. Construction costs for each of these are made up of land cost, site preparation costs, burial costs (where applicable), and erection costs. Land areas for land costs and site preparation costs are computed from the tank capacity and the H/D factor. Costs are summarized in Tables II-29, II-30, and II-32. H/D factors appear in Tables II-26, II27, and II-28. Tank maintenance costs consist of periodic painting and maintenance of metal tanks and are found by the methods described in Section II-F. As in the case of storm water storage, a suboptimization was performed for each case by computing the total system cost for each tank (unless constrained by the instructions described in Section III-B-1-e) as follows:

$$C_k = a(L_k + P_k + B_k + E_k)/365 + O_k \qquad \text{(III-5)}$$

Where: C_K = total system cost of treated water storage facility of configuration k; (dollars/day)

a = capital recovery factor

L_k = Land cost for storage facility of configuration k (dollars);

P_k = Site preparation cost for storage facility of configuration k (dollars);

B_k = Burial cost, where applicable (dollars);

E_k = Erection cost (dollars); and

O_k = Operation and maintenance cost (dollars/day).

The total system costs for the five possible configurations are compared and the least cost alternative is selected and its costs retained. Costs for the other approaches are not retained.

f. Distribution System. Costs for distribution systems for each sub-watershed and each level of reuse are input directly to the computer program. These costs are tabulated in Table II-45. Service areas made up of several watersheds are calculated by adding the appropriate entries from the component sub-watershed costs. The transmission line to the treatment plant is listed separately on Table II-46 and is added to distribution cost. Operation and maintenance cost is a function of total distribution capital cost and is calculated by formula II-40. Total system cost for distribution is calculated as follows:

$$C_d = a(S_d + S_c)/365 + O_d \qquad \text{(III-6)}$$

Where: C_d = total system cost of distribution system (dollars/day);

a = capital recovery factor

S_d = construction cost of distribution system (dollars);

S_c = construction cost of transmission line (dollars); and

O_d = operation and maintenance cost of distribution system (dollars/day).

3. Feasible Systems

a. Possible Collection Areas. Figure III-1 illustrates the flow pattern of the 22 sub-watersheds in the Wilde Lake watershed. As discussed earlier, preliminary investigation revealed that a very large number of combinations of sub-watersheds might be considered as possible collection areas. It was reported that, in order to limit the number of cases to be calculated, no more than six sub-watersheds would be considered a single collection area. Furthermore, many trivial and doubtful cases were neglected and it was found that 110 different collection areas could be defined which would permit 173 combinations for the entire watershed. The collection areas, which are virtually identical with the corresponding service areas, were determined by examination of topographic maps of the watershed and include every case where runoff from one or more sub-watersheds is apparently available at a single place. In each case, that point of collection was identified by the sub-watershed within which it was located and the collection area thus defined was assigned an index number. Tables III-4, III-5, and III-6 list the collection areas identified, showing the sub-watersheds which make up each area, the treatment plant location, and the index number. Each separate table represents a single computer run and is assigned its own set of index numbers. The watershed was divided into three separate areas for analysis designated by Computer Runs A, B, and C, to simplify the computer runs and to make the results of the first analysis available at the earliest possible time, in the event that modifications to the method would be required. The limitation of six sub-watersheds in a single collection area permitted the total watershed to be so divided without any significant penalty in terms of total number of combinations presented for analysis.

b. Combinations of Collection Areas. Having defined all the individual storm water storage, treatment, and reuse facilities possible within the Wilde Lake watershed, subject to the stated constraints, the combinations of these systems which would serve the entire watershed remains to be defined. Again, this may be done by inspection and Tables III-7, III-8, and III-9 list the combinations so identified, defined in terms of index number. Each index number, in turn, refers to the corresponding collection area for the same computer run. In other works, Combination Number 11 in Run B (Table III-8) is made up of Index Numbers 2, 6, and 24, which are sub-watershed numbers 2, 6, and the combination of 1, 3, 11, and 12 (from Table III-5). Each combination represents a portion of a complete system with storage, treatment, and reuse facilities for every area in that section of the watershed. The various methods of assembling the 173 combinations result in 186, 300 ways of serving the entire watershed.

Since each section of the complete watershed that was isolated for a computer run is, by the terms of the assumptions, independent of the others, each can be optimized separately and the optimum system for the watershed will be the summation of the individual optimum systems. This approach reduces the number of cases which must be examined by several orders of magnitude.

Table III-4, Collection Areas - Computer Run A

Index Number	Collection Area (Sub-watersheds)						Treatment Location
1	4						4
2	5						5
3	6						6
4	7						7
5	8						8
6	14						14
7	4	5					4
8	4	14					14
9	5	14					14
10	6	8					6
11	6	7					7
12	7	8					7
13	7	14					14
14	4	5	14				14
15	4	7	14				14
16	5	7	14				14
17	6	7	14				14
18	7	8	14				7
19	6	7	8				7
20	4	5	7				7
21	5	6	7				7
22	5	7	8				7
23	4	6	7				7
24	4	7	8				7
25	4	5	7	14			14
26	4	6	7	14			14
27	5	6	7	14			14
28	4	7	8	14			14
29	5	7	8	14			14
30	6	7	8	14			14
31	4	5	6	7			7
32	4	5	7	8			7
33	4	6	7	8			7
34	5	6	7	8			7
35	4	5	6	8			6
36	4	5	6	7	14		14
37	4	5	7	8	14		14
38	4	6	7	8	14		14
39	5	6	7	8	14		14
40	4	5	6	7	8		7
41	4	5	6	7	8	14	14

Table III-5, Collection Areas - Computer Run B

Index Number	Collection Area (Sub-Watersheds)						Treatment Location
1	1						1
2	2						2
3	3						3
4	11						11
5	12						12
6	13						13
7	1	2					2
8	1	3					3
9	2	3					3
10	3	11					11
11	3	12					12
12	11	12					11
13	11	13					13
14	12	13					13
15	1	2	3				3
16	1	3	11				11
17	1	3	12				12
18	2	3	11				11
19	2	3	12				12
20	3	11	12				11
21	3	11	13				13
22	11	12	13				13
23	1	2	3	11			11
24	1	3	11	12			11
25	1	3	11	13			13
26	2	3	11	12			11
27	2	3	11	13			13
28	3	11	12	13			13
29	1	2	3	11	12		11
30	1	2	3	11	13		13
31	1	3	11	12	13		13
32	2	3	11	12	13		13
33	1	2	3	11	12	13	13

Table III-6, Collection Areas - Computer Run C

Index Number	Collection Area (Sub-watersheds)						Treatment Location
1	15						15
2	16						16
3	17						17
4	18						18
5	9						9
6	20						20
7	10						10
8	19						19
9	21						21
10	22						22
11	15	16					16
12	15	17					17
13	15	18					18
14	16	18					18
15	17	18					18
16	18	9					9
17	9	20					20
18	15	16	17				17
19	15	16	18				18
20	15	17	18				18
21	15	18	9				9
22	16	18	9				9
23	17	18	9				9
24	18	9	20				20
25	15	16	17	18			18
26	15	17	18	9			9
27	15	16	18	9			9
28	15	18	9	20			20
29	16	17	18	9			9
30	16	18	9	20			20
31	17	18	9	20			20
32	15	16	17	18	9		9
33	15	17	18	9	20		20
34	15	16	18	9	20		20
35	16	17	18	9	20		20
36	15	16	17	18	9	20	20

Table III-7, Combination of Areas - Computer Run A

Combination Number	Index Numbers of Collection Areas			Combination Number	Index Numbers of Collection Areas					
1	41			35	19	8	2			
2	40	6		36	19	9	1			
3	39	1		37	19	1	2	6		
4	38	2		38	18	7	3			
5	37	3		39	18	1	2	3		
6	36	5		40	17	7	5			
7	35	13		41	17	1	2	5		
8	35	4	6	42	16	10	1			
9	34	8		43	16	1	3	5		
10	34	1	6	44	15	10	2			
11	33	9		45	15	2	3	5		
12	33	2	6	46	14	11	5			
13	32	3	6	47	14	12	3			
14	31	5	6	48	14	10	4			
15	30	7		49	14	3	4	5		
16	30	1	2	50	13	7	10			
17	29	1	3	51	13	7	3	5		
18	28	2	3	52	13	10	1	2		
19	27	1	5	53	13	1	2	3	5	
20	26	2	5	54	12	7	3	6		
21	25	10		55	12	8	2	3		
22	25	3	5	56	12	9	1	3		
23	24	9	3	57	12	1	2	3	6	
24	24	2	3	6	58	11	7	5	6	
25	23	9	5	59	11	8	2	5		
26	23	2	5	6	60	11	9	1	5	
27	22	8	3	61	11	1	2	5	6	
28	22	1	3	6	62	10	7	4	6	
29	21	8	5	63	10	8	2	4		
30	21	1	5	6	64	10	9	1	4	
31	20	10	6	65	10	1	2	4	6	
32	20	3	5	6	66	9	1	3	4	5
33	19	14		67	8	2	3	4	5	
34	19	7	6	68	7	3	4	5	6	
				69	1	2	3	4	5	6

Table III-8, Combination of Areas - Computer Run B

Combination Number	Index Numbers of Collection Areas				Combination Number	Index Numbers of Collection Areas					
1	33				28	17	2	4	6		
2	32	1			29	16	14	2			
3	31	2			30	16	2	5	6		
4	30	5			31	15	12	6			
5	29	6			32	15	13	5			
6	28	7			33	15	14	4			
7	28	1	2		34	15	4	5	6		
8	27	1	5		35	14	10	7			
9	26	1	6		36	14	9	4	1		
10	25	2	5		37	14	8	4	2		
11	24	2	6		38	14	7	4	3		
12	23	14			39	14	1	2	3	4	
13	23	5	6		40	13	11	7			
14	22	15			41	13	9	1	5		
15	22	7	3		42	13	8	2	5		
16	22	8	2		43	13	7	3	5		
17	22	9	1		44	13	1	2	3	5	
18	22	1	2	3	45	12	7	4	6		
19	21	7	5		46	12	1	2	4	6	
20	21	1	2	5	47	11	7	4	6		
21	20	7	6		48	11	1	2	4	6	
22	20	1	2	6	49	10	7	5	6		
23	19	13	1		50	10	1	2	5	6	
24	19	1	4	6	51	9	1	4	5	6	
25	18	14	1		52	8	2	4	5	6	
26	18	1	5	6	53	7	3	4	5	6	
27	17	13	2		54	6	5	4	3	2	1

Table III-9, Combination of Areas - Computer Run C

Combination Number	Index Numbers of Collection Areas									
1	36	7	8	9	10					
2	35	1	7	8	9	10				
3	34	3	7	8	9	10				
4	33	2	7	8	9	10				
5	32	6	7	8	9	10				
6	31	11	7	8	9	10				
7	31	1	2	7	8	9	10			
8	30	12	7	8	9	10				
9	30	1	3	7	8	9	10			
10	29	1	6	7	8	9	10			
11	28	2	3	7	8	9	10			
12	27	3	6	7	8	9	10			
13	26	2	6	7	8	9	10			
14	25	17	7	8	9	10				
15	25	5	6	7	8	9	10			
16	24	18	7	8	9	10				
17	24	11	3	7	8	9	10			
18	24	12	2	7	8	9	10			
19	24	1	2	3	7	8	9	10		
20	23	11	6	7	8	9	10			
21	23	1	2	6	7	8	9	10		
22	22	12	6	7	8	9	10			
23	22	1	3	6	7	8	9	10		
24	21	2	3	6	7	8	9	10		
25	20	17	2	7	8	9	10			
26	20	2	5	6	7	8	9	10		
27	19	17	3	7	8	9	10			
28	19	3	5	6	7	8	9	10		
29	18	16	6	7	8	9	10			
30	18	17	4	7	8	9	10			
31	18	4	5	6	7	8	9	10		
32	17	11	15	7	8	9	10			
33	17	12	14	7	8	9	10			
34	17	13	2	3	7	8	9	10		
35	17	11	3	4	7	8	9	10		
36	17	12	2	4	7	8	9	10		
37	17	14	1	3	7	8	9	10		
38	17	15	1	2	7	8	9	10		
39	17	1	2	3	4	7	8	9	10	
40	16	11	3	6	7	8	9	10		
41	16	12	2	6	7	8	9	10		
42	16	1	2	3	6	7	8	9	10	
43	15	11	5	6	7	8	9	10		
44	15	1	2	5	6	7	8	9	10	
45	14	12	5	6	7	8	9	10		
46	14	1	3	5	6	7	8	9	10	
47	13	2	3	5	6	7	8	9	10	
48	12	2	4	5	6	7	8	9	10	
49	11	3	4	5	6	7	8	9	10	
50	1	2	3	4	5	6	7	8	9	10

C. System Model Outputs

1. Table of Net Benefits by Case

Section II-D discussed the water demands which are under consideration as possible areas of storm water reuse. It was determined that the four classes of demands could be served in eight different ways: the storm water system could be connected to the "AA, A, B, and C" demands, the "A, B, and C" demands, the "A and B", "A and C", or "B and C" demands, or simply be connected to the "A", "B", or "C" demands independently of the others. In each case, the distribution system and treatment system required is different from any of the other cases. Tables III-4, III-5, and III-6 presented various collection areas that could be considered, each of them made up of one or more sub-watersheds, but permitting storm water to be collected, stored, and treated at a single central location. The first step in the optimization was, therefore, to compute total system costs for each feasible arrangement of facilities within each possible collection area. These results are displayed as a Table of Net Benefits, a sample of which is reproduced as Figure III-2. The case numbers in the left hand column can be observed to correspond to the index numbers of Table III-4. Each line represents a particular collection area, with the sub-watersheds and treatment plant location listed in successive columns. The entries under the eight water service level columns correspond to the net system benefit for the facilities which would serve the demands indicated in the collection area listed. The final column reflects the maximum net benefit appearing on that line and identifies the optimum configuration for that collection area alone. Negative net benefits are identical with net system costs. All values are in dollars/day and are computed as follows:

$$B_n = B_g - C_{jo} - C_p - C_t - C_h - C_{ko} - C_d \qquad \text{(III-7)}$$

Where:

B_n = net system benefits (dollars/day);

B_g = gross system benefit, computed as discussed (See Section III-C-3) (dollars/day);

C_{jo} = sub-optimized system cost of storm water storage from Eq. III-1 (dollars/day);

C_p = system cost of pretreatment from Eq. III-2 (dollars/day);

C_t = system cost of final treatment from Eq. III-3 (dollars/day);

C_h = system cost of pumping from Eq. III-4 (dollars/day);

C_{ko} = sub-optimized system cost of treated water storage from Eq. III-5 (dollars/day); and

C_d = system cost of distribution from Eq. III-6 (dollars/day).

```
                                       ****  TABLE  OF  NET  BENEFITS  ****

CASE        WATERSHED  NUMBER          PLANT                                  -  WATER  QUALITY  -

NUMBER                                 LOCATION       A        B        C       A,B      A,C     A,B,C      B,C  AA,A,B,C      MAX


   1     4  0  0  0  0  0  0  0  0  0      4       -9.18    -7.54    -5.78    -8.60    -8.60    -8.60    -7.20    -6.61    -5.78
   2     5  0  0  0  0  0  0  0  0  0      5      -10.74   -10.09    -7.12    -9.80    -9.17    -9.17    -8.67    -6.52    -6.52
   3     6  0  0  0  0  0  0  0  0  0      6      -41.24   -27.41   -27.72   -41.24   -41.24   -41.24   -31.94    -4.16    -4.16
   4     7  0  0  0  0  0  0  0  0  0      7      -21.72   -16.96   -12.08   -21.72   -21.72   -21.72   -15.83    -4.57    -4.57
   5     8  0  0  0  0  0  0  0  0  0      8      -10.76    -9.07    -6.03    -8.91    -8.88    -8.88    -6.46    -5.26    -5.26
   6    14  0  0  0  0  0  0  0  0  0     14      -19.77   -15.59   -13.47   -16.43   -16.43   -16.43   -11.50    -4.48    -4.48
   7     4  5  0  0  0  0  0  0  0  0      4      -15.58   -12.84   -10.71   -13.43   -13.43   -13.43   -10.21    -5.20    -5.20
   8     4 14  0  0  0  0  0  0  0  0     14      -24.93   -18.65   -17.36   -21.06   -21.06   -21.06   -14.10    -3.55    -3.55
   9     5 14  0  0  0  0  0  0  0  0     14      -25.95   -20.64   -18.14   -21.47   -21.11   -21.11   -15.22    -2.93    -2.93
  10     6  8  0  0  0  0  0  0  0  0      6      -45.92   -31.93   -27.75   -45.92   -45.92   -45.92   -33.86    -1.39    -1.39
  11     6  7  0  0  0  0  0  0  0  0      7      -58.38   -39.64   -37.85   -58.38   -58.38   -58.38   -44.91    -0.14    -0.14
  12     8  7  0  0  0  0  0  0  0  0      7      -26.75   -21.58   -16.35   -26.75   -26.75   -26.75   -16.90    -2.40    -2.40
  13     7 14  0  0  0  0  0  0  0  0     14      -35.28   -28.06   -23.79   -33.97   -33.97   -33.97   -24.02    -1.00    -1.00
  14     4  5 14  0  0  0  0  0  0  0     14      -31.15   -23.69   -22.09   -25.52   -25.52   -25.52   -17.70    -1.54    -1.54
  15     4  7 14  0  0  0  0  0  0  0     14      -40.25   -31.16   -27.75   -38.31   -38.31   -38.31   -28.00     0.53     0.53
  16     5  7 14  0  0  0  0  0  0  0     14      -41.19   -33.13   -28.55   -38.17   -38.17   -38.17   -27.79     1.51     1.51
  17     6  7 14  0  0  0  0  0  0  0     14      -69.22   -50.75   -43.51   -69.22   -69.22   -69.22   -52.99     5.09     5.09
  18     8  7 14  0  0  0  0  0  0  0      7      -41.47   -32.52   -27.93   -38.13   -38.13   -38.13   -27.49     2.71     2.71
  19     6  7  8  0  0  0  0  0  0  0      7      -62.49   -44.21   -37.99   -62.49   -62.49   -62.49   -47.88     3.52     3.52
  20     4  5  7  0  0  0  0  0  0  0      7      -31.02   -25.16   -20.85   -31.02   -31.02   -31.02   -21.07    -1.95    -1.95
  21     6  7  5  0  0  0  0  0  0  0      7      -62.58   -44.71   -39.16   -62.58   -62.58   -62.58   -48.68     2.37     2.37
  22     8  7  5  0  0  0  0  0  0  0      7      -32.32   -26.62   -21.07   -30.95   -30.95   -30.95   -20.67     0.11     0.11
  23     6  7  4  0  0  0  0  0  0  0      7      -62.71   -42.75   -40.31   -62.71   -62.71   -62.71   -48.89     1.39     1.39
  24     8  7  4  0  0  0  0  0  0  0      7      -31.38   -24.63   -20.29   -31.09   -31.09   -31.09   -20.87    -0.87    -0.87
  25     4  5  7 14  0  0  0  0  0  0     14      -46.15   -36.24   -32.55   -42.39   -42.39   -42.39   -31.30     3.09     3.09
  26     4  6  7 14  0  0  0  0  0  0     14      -73.07   -53.86   -46.11   -73.07   -73.07   -73.07   -54.98     6.80     6.80
  27     5  6  7 14  0  0  0  0  0  0     14      -72.63   -55.82   -46.90   -72.63   -72.63   -72.63   -53.54     7.89     7.89
  28     4  8  7 14  0  0  0  0  0  0     14      -46.54   -35.74   -32.03   -42.15   -42.15   -42.15   -29.85     4.29     4.29
  29     5  8  7 14  0  0  0  0  0  0     14      -47.48   -37.70   -32.83   -41.71   -41.71   -41.71   -28.41     5.37     5.37
  30     6  7  8 14  0  0  0  0  0  0     14      -72.39   -55.32   -46.35   -72.39   -72.39   -72.39   -52.10     9.09     9.09
  31     4  5  6  7  0  0  0  0  0  0      7      -66.58   -47.82   -41.57   -66.58   -66.58   -66.58   -51.31     4.02     4.02
  32     4  5  8  7  0  0  0  0  0  0      7      -37.28   -29.69   -25.05   -35.28   -35.28   -35.28   -24.64     1.65     1.65
  33     4  6  7  8  0  0  0  0  0  0      7      -66.34   -47.32   -40.38   -66.34   -66.34   -66.34   -49.87     5.22     5.22
  34     5  6  7  8  0  0  0  0  0  0      7      -65.90   -49.28   -39.44   -65.90   -65.90   -65.90   -48.43     6.31     6.31
  35     4  5  6  8  0  0  0  0  0  0      6      -54.44   -40.11   -33.20   -54.44   -54.44   -54.44   -41.52     2.66     2.66
  36     4  5  6  7 14  0  0  0  0  0     14      -76.47   -58.93   -50.85   -76.47   -76.47   -76.47   -55.52     9.61     9.61
  37     4  5  8  7 14  0  0  0  0  0     14      -52.18   -40.81   -36.83   -45.56   -45.56   -45.56   -30.40     7.08     7.08
  38     4  6  7  8 14  0  0  0  0  0     14      -76.20   -58.77   -50.30   -76.20   -76.20   -76.20   -54.04    10.84    10.84
  39     5  6  7  8 14  0  0  0  0  0     14      -75.70   -60.44   -51.06   -75.70   -75.70   -75.70   -52.54    11.99    11.99
  40     4  5  6  7  8  0  0  0  0  0      7      -69.75   -52.39   -43.39   -69.75   -69.75   -69.75   -50.42     8.02     8.02
  41     4  5  6  7  8 14  0  0  0  0     14      -79.40   -64.12   -54.88   -79.40   -79.40   -79.40   -54.38    13.85    13.85
```

Figure III-2. Table of Net Benefits - Computer Run A

2. Tables of Capital Costs and Daily Costs

Figures III-3 and III-4 show computer listings of capital costs and daily operating and maintenance costs for each of the collection area/reuse level alternatives considered in one of the computer runs. The Table of Capital Cost indicates the total construction cost, in dollars, for each alternative. Case number 11, for example, is a collection area composed of subwatersheds numbers 6 and 7 and for the reuse level "C" (only Class "C" uses considered) has a capital cost of 0.03958E 06, read as 0.03958×10^6, or $39,580. This includes all costs associated with the construction of a storm water storage facility, a pretreatment unit, pumping facilities, treated water storage, and a "C" level distribution system. No final treatment was provided, since only Class "C" water was required. The next entry on the same line, for reuse level "A, B", indicated a capital cost of 0.4658E 06 or $46,580 and includes a Class "A" treatment plant as well as the additional cost of the "A, B" distribution system. Pumping and treated water storage facilities may be sized somewhat differently due to increased total demand.

The Table of Daily Cost, Figure III-4, follows the same format as Figure III-3, except that entries are for total operating and maintenance cost, expressed in dollars/day. The alternative used in the example above, case number 11 at reuse level "C" can be found to have an operating and maintenance cost of $19.70 per day. The "A, B" reuse level for the same case has an operating and maintenance cost of $30.20 per day, largely due to the additional cost of operating the Class "A" treatment plant. These costs include silt removal, maintenance costs of storm water and treated water storage, where applicable, operating and maintenance costs of pretreatment, final treatment, pumping, and distribution facilities. The total system cost of each alternative can be found as follows:

$$C_s = a(C_c)/365 + C_o \qquad \text{(III-8)}$$

Where: C_s = system cost of alternative (dollars/day);

a = capital recovery factor (See page II-4);

C_c = capital cost from Figure III-3 (dollars); and

C_o = daily cost from Figure III-4 (dollars/day).

3. Table of Benefits

The gross system benefits for the alternatives discussed in the preceeding sections were calculated and displayed on a Table of Benefits, shown as Figure III-5. Benefits were taken as the marginal cost from the alternate source of the water used. In this case, benefits were the marginal cost, to the consumer, of an equal volume of water purchased from the Howard County Metropolitan Commission. Under the rate structure in effect in July 1968, that cost is $0.45 per 1000 gallons. The use of this very elementary measure of benefits overlooks the effect of such use on the public water supply system. Many of the costs associated with providing the public water supply to the demand area under study would not be

```
                                       **  TABLE OF CAPITAL COST  **

CASE        WATERSHED  NUMBER            PLANT                                   -  WATER  QUALITY  -

NUMBER                                   LOCATION       A           B           C          A,B         A,C        A,B,C        B,C       AA,A,B,C


   1      4  0  0  0  0  0  0  0  0  0     4    0.7069E 050.6118E 050.5139E 050.7100E 050.7100E 050.7100E 050.6982F 050.5752E 05
   2      5  0  0  0  0  0  0  0  0  0     5    0.8307E 050.6915E 050.6231E 050.8357E 050.8391E 050.8391E 050.8081E 050.6405E 05
   3      6  0  0  0  0  0  0  0  0  0     6    0.3145E 060.2060E 060.2642E 060.3145E 060.3145E 060.3145E 060.2835E 060.7175E 05
   4      7  0  0  0  0  0  0  0  0  0     7    0.1800E 060.1233E 060.1454E 060.1800E 060.1800E 060.1800E 060.1644E 060.6493E 05
   5      8  0  0  0  0  0  0  0  0  0     8    0.8976E 050.6836E 050.6609E 050.9073E 050.9075E 050.9075F 050.8495E 050.6257E 05
   6     14  0  0  0  0  0  0  0  0  0    14    0.1505E 060.1227E 060.1246E 060.1528E 060.1528E 060.1528E 060.1446F 060.6894E 05
   7      4  5  0  0  0  0  0  0  0  0     4    0.1235E 060.9704E 050.9845E 050.1248E 060.1248E 060.1248E 060.1175E 060.6644E 05
   8      4 14  0  0  0  0  0  0  0  0    14    0.1933E 060.1527E 060.1629E 060.1963E 060.1963E 060.1963E 060.1839E 060.7394F 05
   9      5 14  0  0  0  0  0  0  0  0    14    0.2019E 060.1567E 060.1700E 060.2053E 060.2056E 060.2056E 060.1910E 060.7686E 05
  10      6  8  0  0  0  0  0  0  0  0     6    0.3779E 060.2427E 060.3181E 060.3779E 060.3779E 060.3779E 060.3382E 060.8194E 05
  11      6  7  0  0  0  0  0  0  0  0     7    0.4658E 060.2964E 060.3958E 060.4658E 060.4658E 060.4658E 060.4161E 060.8293E 05
  12      8  7  0  0  0  0  0  0  0  0     7    0.2441E 060.1607E 060.1992E 060.2441E 060.2441E 060.2441F 060.2199F 060.7586E 05
  13      7 14  0  0  0  0  0  0  0  0    14    0.3051E 060.2148E 060.2578E 060.3060E 060.3060E 060.3060E 060.2792E 06C.8203E 05
  14      4  5 14  0  0  0  0  0  0  0    14    0.2449E 060.1867E 060.2086E 060.2490E 060.2490E 060.2490E 060.2305E 060.8181E 05
  15      4  7 14  0  0  0  0  0  0  0    14    0.3481E 060.2451E 060.2964E 060.3494E 060.3494E 060.3494E 060.3184E 060.8698E 05
  16      5  7 14  0  0  0  0  0  0  0    14    0.3567E 060.2489E 060.3035E 060.3587E 060.3587E 060.3587E 060.3261E 060.8989E 05
  17      6  7 14  0  0  0  0  0  0  0    14    0.5925E 060.3880E 060.5086E 060.5925E 060.5925E 060.5925E 060.5302E 060.1031E 06
  18      8  7 14  0  0  0  0  0  0  0     7    0.3663E 060.2510E 060.3102E 060.3686E 060.3686E 060.3686E 060.3332E 060.9143E 05
  19      6  7  8  0  0  0  0  0  0  0     7    0.5294E 060.3335E 060.4497E 060.5294E 060.5294E 060.5294E 060.4708E 060.9397E 05
  20      4  5  7  0  0  0  0  0  0  0     7    0.2769E 060.1881E 060.2304E 060.2769E 060.2769E 060.2769E 060.2513E 060.7852E 05
  21      6  7  5  0  0  0  0  0  0  0     7    0.5185E 060.3306E 060.4421E 060.5185E 060.5185E 060.5185E 060.4630E 060.9080E 05
  22      8  7  5  0  0  0  0  0  0  0     7    0.2959E 060.1947E 060.2449E 060.2968E 060.2968E 060.2968E 060.2668E 060.8372E 05
  23      6  7  4  0  0  0  0  0  0  0     7    0.5092E 060.3268E 060.4347E 060.5092E 060.5092E 060.5092E 060.4554E 060.8788E 05
  24      8  7  4  0  0  0  0  0  0  0     7    0.2873E 060.1907E 060.2378E 060.2875E 060.2875E 060.2875E 060.2591E 060.8081E 05
  25      4  5  7 14  0  0  0  0  0  0    14    0.3997E 060.2793E 060.3421E 060.4024E 060.4024E 060.4024E 060.3654E 060.9551E 05
  26      4  6  7 14  0  0  0  0  0  0    14    0.6370E 060.4183E 060.5474E 060.6370E 060.6370E 060.6370E 060.5695E 060.1109E 06
  27      5  6  7 14  0  0  0  0  0  0    14    0.6470E 060.4222E 060.5545E 060.6470E 060.6470E 060.6470E 060.5772E 060.1156E 06
  28      4  8  7 14  0  0  0  0  0  0    14    0.4101E 060.2822E C60.3496E 060.4137E 060.4137E 060.4137E 060.3732E 060.9962E 05
  29      5  8  7 14  0  0  0  0  0  0    14    0.4186E 060.2860E 060.3566E 060.4237E 060.4237E 060.4237E 060.3809E 060.1043E 06
  30      6  7  8 14  0  0  0  0  0  0    14    0.6583E 060.4250E 060.5619E 060.6583E 060.6583E 060.6583E 060.5851E 060.1197E 06
  31      4  5  6  7  0  0  0  0  0  0     7    0.5626E 060.3610E 060.4810E 060.5626E 060.5626E 060.5626E 060.5023E 060.9769E 05
  32      4  5  8  7  0  0  0  0  0  0     7    0.3389E 060.2248F 060.2835E 060.3403E 060.3403E 060.3403E 060.3060E 060.8867E 05
  33      4  6  7  8  0  0  0  0  0  0     7    0.5740E 060.3639E 060.4885E 060.5740E 060.5740E 060.5740E 060.5102E 060.1018E 06
  34      5  6  7  8  0  0  0  0  0  0     7    0.5840E 060.3677E 060.4959E 060.5840E 060.5840E 060.5840E 060.5178E 060.1065E 06
  35      4  5  6  8  0  0  0  0  0  0     6    0.4741E 060.3073E 060.4030E 060.4741E 060.4741E 060.4741E 060.4244E 060.9487E 05
  36      4  5  6  7 14  0  0  0  0  0    14    0.6914E 060.4525E 060.5930E 060.6914E 060.6914E 060.6914E 060.6164E 060.1233E 06
  37      4  5  8  7 14  0  0  0  0  0    14    0.4622E 060.3164E 060.3952E 060.4682E 060.4682E 060.4682E 060.4202E 060.1121E 06
  38      4  6  7  8 14  0  0  0  0  0    14    0.7025E 060.4554E 060.6004E 060.7025E 060.7025E 060.7025E 060.6241E 060.1272E 06
  39      5  6  7  8 14  0  0  0  0  0    14    0.7121E 060.4590E 060.6073E 060.7121E 060.7121E 060.7121E 060.6314E 060.1315E 06
  40      4  5  6  7  8  0  0  0  0  0     7    0.6285E 060.3981E 060.5345E 060.6285E 060.6285E 060.6285E 060.5572E 060.1143E 06
  41      4  5  6  7  8 14  0  0  0  0    14    0.7556E 060.4887E 060.6449E 060.7556E 060.7556E 060.7556E 060.6[illegible]6E 060.1382E 06
```

Figure III-3. Table of Capital Cost - Computer Run A

```
                              ** TABLE OF DAILY COST **

CASE      WATERSHED  NUMBER          PLANT                          -  WATER  QUALITY  -

NUMBER                              LOCATION       A        B        C       A,B      A,C     A,B,C      B,C   AA,A,B,C

   1     4  0  0  0  0  0  0  0  0  0      4      3.63     2.16     2.15     4.27     4.27     4.27     3.04     4.21
   2     5  0  0  0  0  0  0  0  0  0      5      4.91     2.21     2.89     5.93     6.62     6.62     3.82     6.82
   3     6  0  0  0  0  0  0  0  0  0      6     17.84     8.36    11.54    17.84    17.84    17.84    12.99    15.57
   4     7  0  0  0  0  0  0  0  0  0      7     12.81     5.19     8.00    12.81    12.81    12.81     9.16    12.17
   5     8  0  0  0  0  0  0  0  0  0      8      7.00     3.26     4.09     9.01     9.05     9.05     5.79     9.47
   6    14  0  0  0  0  0  0  0  0  0     14      9.93     5.96     6.44    13.63    13.63    13.63     9.57    13.72
   7     4  5  0  0  0  0  0  0  0  0      4      8.53     4.37     5.03    10.88    10.88    10.88     7.24    11.02
   8     4 14  0  0  0  0  0  0  0  0     14     13.55     8.11     8.58    17.88    17.88    17.88    12.69    17.91
   9     5 14  0  0  0  0  0  0  0  0     14     14.82     8.16     9.32    19.83    20.23    20.23    13.37    20.52
  10     6  8  0  0  0  0  0  0  0  0      6     26.62    11.61    17.02    26.62    26.62    26.62    20.24    24.53
  11     6  7  0  0  0  0  0  0  0  0      7     30.20    13.54    19.70    30.20    30.20    30.20    23.85    26.86
  12     8  7  0  0  0  0  0  0  0  0      7     21.83     8.43    12.07    21.83    21.83    21.83    15.46    21.61
  13     7 14  0  0  0  0  0  0  0  0     14     24.93    11.13    14.42    26.11    26.11    26.11    20.01    25.26
  14     4  5 14  0  0  0  0  0  0  0     14     18.44    10.31    11.46    24.26    24.26    24.26    17.32    24.27
  15     4  7 14  0  0  0  0  0  0  0     14     28.32    13.28    16.59    30.07    30.07    30.07    24.21    28.87
  16     5  7 14  0  0  0  0  0  0  0     14     29.52    13.33    17.34    32.24    32.24    32.24    26.54    31.11
  17     6  7 14  0  0  0  0  0  0  0     14     41.99    19.48    28.30    41.99    41.99    41.99    34.68    37.86
  18     8  7 14  0  0  0  0  0  0  0      7     31.46    14.38    18.58    34.48    34.48    34.48    28.92    33.39
  19     6  7  8  0  0  0  0  0  0  0      7     38.36    16.78    25.29    38.36    38.36    38.36    32.15    34.81
  20     4  5  7  0  0  0  0  0  0  0      7     23.56     9.54    13.02    23.56    23.56    23.56    17.27    22.94
  21     6  7  5  0  0  0  0  0  0  0      7     36.32    15.74    23.85    36.32    36.32    36.32    30.38    32.71
  22     8  7  5  0  0  0  0  0  0  0      7     26.72    10.64    14.95    27.96    27.96    27.96    21.99    27.46
  23     6  7  4  0  0  0  0  0  0  0      7     34.16    15.69    22.43    34.16    34.16    34.16    28.05    30.47
  24     8  7  4  0  0  0  0  0  0  0      7     25.53    10.59    14.21    25.79    25.79    25.79    19.66    25.22
  25     4  5  7 14  0  0  0  0  0  0     14     32.91    15.48    19.54    36.05    36.05    36.05    30.27    34.59
  26     4  6  7 14  0  0  0  0  0  0     14     45.30    21.63    30.90    45.30    45.30    45.30    36.89    40.89
  27     5  6  7 14  0  0  0  0  0  0     14     47.07    21.68    31.65    47.07    47.07    47.07    37.98    42.76
  28     4  8  7 14  0  0  0  0  0  0     14     34.85    16.53    20.78    37.88    37.88    37.88    31.39    36.49
  29     5  8  7 14  0  0  0  0  0  0     14     36.04    16.58    21.53    39.65    39.65    39.65    32.49    38.37
  30     6  7  8 14  0  0  0  0  0  0     14     48.90    22.72    32.86    48.90    48.90    48.90    39.11    44.67
  31     4  5  6  7  0  0  0  0  0  0      7     39.85    17.89    26.55    39.85    39.85    39.85    33.23    35.92
  32     4  5  8  7  0  0  0  0  0  0      7     30.11    12.79    17.13    31.92    31.92    31.92    26.19    31.07
  33     4  6  7  8  0  0  0  0  0  0      7     41.68    18.93    27.97    41.68    41.68    41.68    34.36    37.83
  34     5  6  7  8  0  0  0  0  0  0      7     43.44    18.98    29.27    43.44    43.44    43.44    35.45    39.70
  35     4  5  6  8  0  0  0  0  0  0      6     36.68    15.96    23.33    36.68    36.68    36.68    30.89    33.97
  36     4  5  6  7 14  0  0  0  0  0     14     50.39    23.83    33.81    50.39    50.39    50.39    40.19    45.79
  37     4  5  8  7 14  0  0  0  0  0     14     39.08    18.73    23.73    42.96    42.96    42.96    34.69    41.39
  38     4  6  7  8 14  0  0  0  0  0     14     52.21    25.22    35.02    52.21    52.21    52.21    41.31    47.69
  39     5  6  7  8 14  0  0  0  0  0     14     53.98    25.00    35.76    53.98    53.98    53.98    42.40    49.57
  40     4  5  6  7  8  0  0  0  0  0      7     46.76    21.13    31.44    46.76    46.76    46.76    37.66    42.73
  41     4  5  6  7  8 14  0  0  0  0     14     57.29    27.83    37.93    57.29    57.29    57.29    44.61    52.59
```

Figure III-4. Table of Daily Cost - Computer Run A

```
                                               ** TABLE OF BENEFITS **

CASE        WATERSHED  NUMBER             PLANT                                   - WATER  QUALITY -

NUMBER                                    LOCATION       A         B         C       A,B       A,C     A,B,C       B,C  AA,A,B,C

   1      4  0  0  0  0  0  0  0  0  0       4        4.59      3.40      3.74      5.85      5.85      5.85      5.85      5.85
   2      5  0  0  0  0  0  0  0  0  0       5        6.07      2.04      4.70      8.11      9.49      9.49      6.74      9.49
   3      6  0  0  0  0  0  0  0  0  0       6       21.71     10.49     21.71     21.71     21.71     21.71     21.71     21.71
   4      7  0  0  0  0  0  0  0  0  0       7       16.91      5.91     16.76     16.91     16.91     16.91     16.91     16.91
   5      8  0  0  0  0  0  0  0  0  0       8        9.11      3.99      7.53     13.10     13.18     13.18     11.52     13.18
   6     14  0  0  0  0  0  0  0  0  0      14       11.75      7.95     10.84     19.12     19.12     19.12     18.80     19.12
   7      4  5  0  0  0  0  0  0  0  0       4       10.66      5.44      8.44     15.34     15.34     15.34     13.88     15.34
   8      4 14  0  0  0  0  0  0  0  0      14       16.34     11.35     14.58     24.97     24.97     24.97     24.97     24.97
   9      5 14  0  0  0  0  0  0  0  0      14       17.82      9.99     15.55     27.82     28.61     28.61     25.54     28.61
  10      6  8  0  0  0  0  0  0  0  0       6       34.89     14.48     34.89     34.89     34.89     34.89     34.89     34.89
  11      6  7  0  0  0  0  0  0  0  0       7       38.61     16.41     38.61     38.61     38.61     38.61     38.61     38.61
  12      8  7  0  0  0  0  0  0  0  0       7       30.09      9.90     24.29     30.09     30.09     30.09     30.09     30.09
  13      7 14  0  0  0  0  0  0  0  0      14       33.40     13.87     27.61     36.03     36.03     36.03     36.03     36.03
  14      4  5 14  0  0  0  0  0  0  0      14       22.41     13.39     19.29     34.46     34.46     34.46     32.68     34.46
  15      4  7 14  0  0  0  0  0  0  0      14       37.99     17.27     31.35     41.88     41.88     41.88     41.88     41.88
  16      5  7 14  0  0  0  0  0  0  0      14       39.48     15.91     32.31     45.52     45.52     45.52     45.52     45.52
  17      6  7 14  0  0  0  0  0  0  0      14       57.73     24.36     57.73     57.73     57.73     57.73     57.73     57.73
  18      8  7 14  0  0  0  0  0  0  0       7       42.52     17.86     35.14     49.21     49.21     49.21     49.21     49.21
  19      6  7  8  0  0  0  0  0  0  0       7       51.80     20.40     51.80     51.80     51.80     51.80     51.80     51.80
  20      4  5  7  0  0  0  0  0  0  0       7       32.25     11.35     25.21     32.25     32.25     32.25     32.25     32.25
  21      6  7  5  0  0  0  0  0  0  0       7       48.10     18.45     48.10     48.10     48.10     48.10     48.10     48.10
  22      8  7  5  0  0  0  0  0  0  0       7       36.84     11.94     29.00     39.58     39.58     39.58     39.58     39.58
  23      6  7  4  0  0  0  0  0  0  0       7       44.47     19.81     44.47     44.47     44.47     44.47     44.47     44.47
  24      8  7  4  0  0  0  0  0  0  0       7       35.36     13.30     28.03     35.94     35.94     35.94     35.94     35.94
  25      4  5  7 14  0  0  0  0  0  0      14       44.07     19.31     36.05     51.37     51.37     51.37     51.37     51.37
  26      4  6  7 14  0  0  0  0  0  0      14       63.59     27.76     63.30     63.59     63.59     63.59     63.59     63.59
  27      5  6  7 14  0  0  0  0  0  0      14       67.22     26.40     64.27     67.22     67.22     67.22     67.22     67.22
  28      4  8  7 14  0  0  0  0  0  0      14       47.11     21.26     38.88     55.06     55.06     55.06     55.06     55.06
  29      5  8  7 14  0  0  0  0  0  0      14       48.59     19.90     39.84     58.70     58.70     58.70     58.70     58.70
  30      6  7  8 14  0  0  0  0  0  0      14       70.92     28.35     67.09     70.92     70.92     70.92     70.92     70.92
  31      4  5  6  7  0  0  0  0  0  0       7       53.96     21.84     53.96     53.96     53.96     53.96     53.96     53.96
  32      4  5  8  7  0  0  0  0  0  0       7       41.43     15.34     32.74     45.43     45.43     45.43     45.43     45.43
  33      4  6  7  8  0  0  0  0  0  0       7       57.65     23.80     57.65     57.65     57.65     57.65     57.65     57.65
  34      5  6  7  8  0  0  0  0  0  0       7       61.29     22.44     60.95     61.29     61.29     61.29     61.29     61.29
  35      4  5  6  8  0  0  0  0  0  0       6       50.23     19.92     47.93     50.23     50.23     50.23     50.23     50.23
  36      4  5  6  7 14  0  0  0  0  0      14       73.08     29.80     68.01     73.08     73.08     73.08     73.08     73.08
  37      4  5  8  7 14  0  0  0  0  0      14       53.18     23.30     43.58     64.55     64.55     64.55     64.55     64.55
  38      4  6  7  8 14  0  0  0  0  0      14       76.77     31.75     70.83     76.77     76.77     76.77     76.77     76.77
  39      5  6  7  8 14  0  0  0  0  0      14       80.41     30.39     71.80     80.41     80.41     80.41     80.41     80.41
  40      4  5  6  7  8  0  0  0  0  0       7       67.14     25.83     64.69     67.14     67.14     67.14     67.14     67.14
  41      4  5  6  7  8 14  0  0  0  0      14       86.26     33.79     75.54     86.26     86.26     86.26     86.26     86.26
```

Figure III-5. Table of Benefits - Computer Run A

reduced by the introduction of the supplementary source. The distribution system, the cost of billing and collection, the overhead structure of the Metropolitan Commission - all of these would be essentially unchanged. Since all of the water distributed by Howard County is purchased from Baltimore City and the Washington Suburban Sanitary Commission, the most clear cut reduction in true costs would be the reduction in the quantity of water purchased, which accounts for less than half of the total selling price of water. Other costs would be reduced in a less easily determined manner, such as booster pumping costs, storage costs, transmission main costs, etc. To properly evaluate the total impact of the study system on the public supply system, a detailed study of the financial structure of the Metropolitan Commission would be required, a task outside the scope of this work. Since the Wilde Lake area constitutes a small portion of the county water distribution area, it was considered that the elimination of a portion of the demand in this watershed would not have a significant overall effect on the cost of supplying water to the county. For this reason, the full price of water was taken as a benefit, although in a proposed large-scale application of the local reuse concept, this assumption would require careful review in the light of local circumstances. Ideally, the economic benefit due to water supply should be taken as the marginal reduction in the total long-term costs of the public water system, due to the reduction in demand on that system.

At the time the computer runs were performed, the marginal cost of water in Howard County had recently been revised to \$0.50 per 1000 gallons. Shortly after the runs were completed, however, the rate was reduced to \$0.45 per 1000 gallons. Accordingly, Figure III-5 shows benefits calculated on the basis of the higher figure, and subsequent discussion of net benefits in this chapter will be in terms of \$0.50 per 1000 gallons marginal cost of water. Prior to the definition of the conceptual optimum systems described in Chapter IV, however, and prior to the system evaluation reported in Chapter VI, the benefit rate was revised to \$0.45 per 1000 gallons and economic comparisons made on that basis. Since the benefit is, in this case, completely linear, and independent of other assumptions, manipulating the rate has no effect on system optima. The individual benefits listed on Figure III-5 are calculated as follows:

$$B_g = (Q_N)(r) \tag{III-9}$$

Where:
B_g = gross system benefits of alternative (dollars/day);
Q_N = treated water demand of alternative (annual mean - gal/day); and
r = benefit rate, marginal cost of water to consumer from public system (dollars/gal).

An alternative method of calculating the net system benefits displayed on Figure III-2 and defined by Equation III-7 is presented below:

$$B_n = B_g - C_s \tag{III-10}$$

Where:
B_n = net system benefits (dollars/day);
B_g = gross system benefits from Eq. III-9 (dollars/day); and
C_s = system cost from Eq. III-8 (dollars/day).

4. Table of Net Benefits for Combinations of Watersheds

Tables III-7, III-8, and III-9, described earlier, defined the various combinations of collection areas which would satisfy the entire section of the watershed under consideration. The entire watershed was analyzed in three sections, each the subject of a single computer run. In order to compare and rank the various alternative systems which would serve the section under study, it is necessary, for each combination of collection areas, to add the net benefits associated with the most attractive reuse levels for each of the component collection areas, obtaining the total net benefit for the combination. This is done by adding the net benefits appearing in the final column of the Table of Net Benefits (Figure III-2) for the collection areas which make up a given combination. When this is done for each possible combination, the results can be ranked and the optimum system for that section identified.

In order to obtain as much information about alternative strategies as possible, a five-level output was obtained. This output defined optimum systems under each of the following assumptions:

(1) All levels of reuse possible;
(2) Only reuse up to water quality "A" possible;
(3) Only reuse up to water quality "B" possible;
(4) Only reuse up to water quality "C" possible; and
(5) No reuse possible.

This output for computer run A is displayed as Figures III-6 and III-7. Figure III-6 lists total net benefits for all possible combinations under the first four assumptions noted above. Column "AA" reflects combinations of optimum sub-systems when all levels of reuse are possible. It should be noted that the optimum sub-systems might be composed of any assortment of reuses. The only requirement is that each collection area be served by the system having the maximum net benefit for that area. These sub-systems are those identified by the MAX column on Figure III-2. Column "A" follows the same pattern, except that all sub-systems involving the use of "AA" water and "A" are excluded from the analysis. Column "B" excludes all "AA" uses and Column "C" excludes all but "C." The assumption of no reuse required a separate computer run, due to the limitation of the system to storm water storage and pretreatment facilities. Since no benefits, as defined in the economic analysis, derive from this application, the net benefits reported are identical with system cost. Figure III-7 lists the totals for the various combinations. This assumption describes a system which retains only water pollution control as its purpose and permits direct comparison with the alternative method of water pollution control described in later chapters.

```
                 ****  TABLE  OF  NET  BENEFITS  FOR  COMBINATIONS  OF  WATERSHEDS  ****

COMBINATION          PLANT COMBINATIONS,  PERMISSIBLE  MAXIMUM  QUALITY

  NUMBER                                                         AA          A           B           C


    1        41  0  0  0  0  0  0  0  0  0                     13.85      -54.38      -54.38      -54.88
    2        40  6  0  0  0  0  0  0  0  0                      3.54      -54.90      -54.90      -56.86
    3        39  1  0  0  0  0  0  0  0  0                      6.20      -56.84      -56.84      -56.84
    4        38  2  0  0  0  0  0  0  0  0                      4.33      -57.42      -57.42      -57.42
    5        37  3  0  0  0  0  0  0  0  0                      2.93      -57.82      -57.82      -64.55
    6        36  5  0  0  0  0  0  0  0  0                      4.35      -56.88      -56.88      -56.88
    7        35 13  0  0  0  0  0  0  0  0                      1.66      -57.00      -57.00      -57.00
    8        35  4  6  0  0  0  0  0  0  0                     -6.40      -56.79      -56.79      -58.75
    9        34  8  0  0  0  0  0  0  0  0                      2.77      -53.54      -53.54      -56.80
   10        34  1  6  0  0  0  0  0  0  0                     -3.96      -56.72      -56.72      -58.69
   11        33  9  0  0  0  0  0  0  0  0                      2.29      -55.60      -55.60      -58.53
   12        33  2  6  0  0  0  0  0  0  0                     -5.78      -59.00      -59.00      -60.97
   13        32  3  6  0  0  0  0  0  0  0                     -6.99      -63.55      -63.55      -66.24
   14        31  5  6  0  0  0  0  0  0  0                     -5.72      -59.11      -59.11      -61.08
   15        30  7  0  0  0  0  0  0  0  0                      3.88      -56.56      -56.56      -57.06
   16        30  1  2  0  0  0  0  0  0  0                     -3.21      -59.25      -59.25      -59.25
   17        29  1  3  0  0  0  0  0  0  0                     -4.57      -61.61      -61.61      -66.34
   18        28  2  3  0  0  0  0  0  0  0                     -6.39      -64.39      -64.39      -66.88
   19        27  1  5  0  0  0  0  0  0  0                     -3.15      -58.71      -58.71      -58.71
   20        26  2  5  0  0  0  0  0  0  0                     -4.97      -59.26      -59.26      -59.26
   21        25 10  0  0  0  0  0  0  0  0                      1.69      -59.05      -59.05      -60.30
   22        25  3  5  0  0  0  0  0  0  0                     -6.33      -64.74      -64.74      -66.30
   23        24  9  3  0  0  0  0  0  0  0                     -7.95      -62.92      -62.92      -66.15
   24        24  2  3  6  0  0  0  0  0  0                    -16.02      -66.32      -66.32      -68.60
   25        23  9  5  0  0  0  0  0  0  0                     -6.80      -61.56      -61.56      -64.48
   26        23  2  5  6  0  0  0  0  0  0                    -14.86      -64.96      -64.96      -66.93
   27        22  8  3  0  0  0  0  0  0  0                     -7.59      -62.18      -62.18      -66.15
   28        22  1  3  6  0  0  0  0  0  0                    -14.31      -65.36      -65.36      -68.05
   29        21  8  5  0  0  0  0  0  0  0                     -6.43      -59.29      -59.29      -62.55
   30        21  1  5  6  0  0  0  0  0  0                    -13.15      -62.48      -62.48      -64.45
   31        20 10  6  0  0  0  0  0  0  0                     -7.83      -60.11      -60.11      -62.07
   32        20  3  5  6  0  0  0  0  0  0                    -15.85      -65.80      -65.80      -68.07
   33        19 14  0  0  0  0  0  0  0  0                      1.98      -55.69      -55.69      -60.08
   34        19  7  6  0  0  0  0  0  0  0                     -6.17      -59.70      -59.70      -62.17
   35        19  8  2  0  0  0  0  0  0  0                     -6.55      -59.21      -59.21      -62.47
   36        19  9  1  0  0  0  0  0  0  0                     -5.20      -58.99      -58.99      -61.92
   37        19  1  2  6  0  0  0  0  0  0                    -13.27      -62.40      -62.40      -64.36
   38        18  7  3  0  0  0  0  0  0  0                     -6.65      -65.11      -65.11      -66.36
   39        18  1  2  3  0  0  0  0  0  0                    -13.75      -67.80      -67.80      -68.55
   40        17  7  5  0  0  0  0  0  0  0                     -5.37      -59.75      -59.75      -60.25
   41        17  1  2  5  0  0  0  0  0  0                    -12.46      -62.45      -62.45      -62.45
   42        16 10  1  0  0  0  0  0  0  0                     -5.66      -61.33      -61.33      -62.09
   43        16  1  3  5  0  0  0  0  0  0                    -13.68      -67.02      -67.02      -68.09
   44        15 10  2  0  0  0  0  0  0  0                     -7.37      -62.63      -62.63      -62.63
   45        15  2  3  5  0  0  0  0  0  0                    -15.40      -68.32      -68.32      -68.63
   46        14 11  5  0  0  0  0  0  0  0                     -6.94      -61.58      -61.58      -65.97
   47        14 12  3  0  0  0  0  0  0  0                     -8.09      -61.46      -61.46      -66.16
   48        14 10  4  0  0  0  0  0  0  0                     -7.51      -57.54      -57.54      -61.92
   49        14  3  4  5  0  0  0  0  0  0                    -15.53      -63.23      -63.23      -67.92
   50        13  7 10  0  0  0  0  0  0  0                     -7.60      -61.76      -61.76      -62.26
   51        13  7  3  5  0  0  0  0  0  0                    -15.62      -67.45      -67.45      -68.26
   52        13 10  1  2  0  0  0  0  0  0                    -14.69      -64.45      -64.45      -64.45



   53        13  1  2  3  5  0  0  0  0  0                    -22.71      -70.14      -70.14      -70.45
   54        12  7  3  6  0  0  0  0  0  0                    -16.24      -65.47      -65.47      -68.25
   55        12  8  2  3  0  0  0  0  0  0                    -16.61      -64.98      -64.98      -68.55
   56        12  9  1  3  0  0  0  0  0  0                    -15.27      -64.76      -64.76      -68.00
   57        12  1  2  3  6  0  0  0  0  0                    -23.34      -68.17      -68.17      -70.44
   58        11  7  5  6  0  0  0  0  0  0                    -15.09      -65.59      -65.59      -68.06
   59        11  8  2  5  0  0  0  0  0  0                    -15.46      -65.10      -65.10      -68.36
   60        11  9  1  5  0  0  0  0  0  0                    -14.11      -64.88      -64.88      -67.81
   61        11  1  2  5  6  0  0  0  0  0                    -22.18      -68.29      -68.29      -70.25
   62        10  7  4  6  0  0  0  0  0  0                    -15.66      -61.55      -61.55      -64.02
   63        10  8  2  4  0  0  0  0  0  C                    -16.03      -61.06      -61.06      -64.32
   64        10  9  1  4  0  0  0  0  0  0                    -14.68      -60.84      -60.84      -63.76
   65        10  1  2  4  6  0  0  0  0  0                    -22.75      -64.24      -64.24      -66.21
   66         9  1  3  4  5  0  0  0  0  0                    -22.70      -66.53      -66.53      -69.76
   67         8  2  3  4  5  0  0  0  0  0                    -24.05      -66.75      -66.75      -70.32
   68         7  3  4  5  6  0  0  0  0  0                    -23.68      -67.24      -67.24      -70.02
   69         1  2  3  4  5  6  0  0  0  0                    -30.77      -69.93      -69.93      -72.21
```

Figure III-6. Table of Net Benefits of Combinations of Watersheds - Reuse Computer Run A

```
                    ****  TABLE  OF  NET  BENEFITS  FOR  COMBINATIONS  OF  WATERSHEDS  ****

~~COMBINATION          PLANT COMBINATIONS,  PERMISSIBLE  MAXIMUM  QUALITY~~

   NUMBER
                                                                   NO REUSE

     1          41  0  0  0  0  0  0  0  0  0                      -15.99
~~   2          40  6  0  0  0  0  0  0  0  0                      -17.14~~
     3          39  1  0  0  0  0  0  0  0  0                      -16.80
     4          38  2  0  0  0  0  0  0  0  0                      -16.92
     5          37  3  0  0  0  0  0  0  0  0                      -17.14
     6          36  5  0  0  0  0  0  0  0  0                      -16.96
     7          35 13  0  0  0  0  0  0  0  0                      -17.12
~~   8          35  4  6  0  0  0  0  0  0  0                      -18.08~~
     9          34  8  0  0  0  0  0  0  0  0                      -17.14
    10          34  1  6  0  0  0  0  0  0  0                      -17.86
    11          33  9  0  0  0  0  0  0  0  0                      -17.13
    12          33  2  6  0  0  0  0  0  0  0                      -17.95
    13          32  3  6  0  0  0  0  0  0  0                      -18.17
~~  14          31  5  6  0  0  0  0  0  0  0                      -18.08~~
    15          30  7  0  0  0  0  0  0  0  0                      -17.09
    16          30  1  2  0  0  0  0  0  0  0                      -17.63
    17          29  1  3  0  0  0  0  0  0  0                      -17.85
    18          28  2  3  0  0  0  0  0  0  0                      -17.95
    19          27  1  5  0  0  0  0  0  0  0                      -17.67
~~  20          26  2  5  0  0  0  0  0  0  0                      -17.77~~
    21          25 10  0  0  0  0  0  0  0  0                      -17.12
    22          25  3  5  0  0  0  0  0  0  0                      -17.99
    23          24  9  3  0  0  0  0  0  0  0                      -18.16
    24          24  2  3  6  0  0  0  0  0  0                      -18.98
    25          23  9  5  0  0  0  0  0  0  0                      -17.98
~~  26          23  2  5  6  0  0  0  0  0  0                      -18.80~~
    27          22  8  3  0  0  0  0  0  0  0                      -18.16
    28          22  1  3  6  0  0  0  0  0  0                      -18.88
    29          21  8  5  0  0  0  0  0  0  0                      -17.99
    30          21  1  5  6  0  0  0  0  0  0                      -18.71
    31          20 10  6  0  0  0  0  0  0  0                      -18.16
~~  32          20  3  5  6  0  0  0  0  0  0                      -19.03~~
    33          19 14  0  0  0  0  0  0  0  0                      -17.12
    34          19  7  6  0  0  0  0  0  0  0                      -18.12
    35          19  8  2  0  0  0  0  0  0  0                      -17.94
    36          19  9  1  0  0  0  0  0  0  0                      -17.84
    37          19  1  2  6  0  0  0  0  0  0                      -18.66
~~  38          18  7  3  0  0  0  0  0  0  0                      -18.11~~
    39          18  1  2  3  0  0  0  0  0  0                      -18.66
    40          17  7  5  0  0  0  0  0  0  0                      -17.94
    41          17  1  2  5  0  0  0  0  0  0                      -18.48
    42          16 10  1  0  0  0  0  0  0  0                      -17.83
    43          16  1  3  5  0  0  0  0  0  0                      -18.70
~~  44          15 10  2  0  0  0  0  0  0  0                      -17.93~~
    45          15  2  3  5  0  0  0  0  0  0                      -18.80
    46          14 11  5  0  0  0  0  0  0  0                      -17.97
    47          14 12  3  0  0  0  0  0  0  0                      -18.16
    48          14 10  4  0  0  0  0  0  0  0                      -18.06
    49          14  3  4  5  0  0  0  0  0  0                      -18.93
~~  50          13  7 10  0  0  0  0  0  0  0                      -18.10~~
    51          13  7  3  5  0  0  0  0  0  0                      -18.96
    52          13 10  1  2  0  0  0  0  0  0                      -18.64
    53          13  1  2  3  5  0  0  0  0  0                      -19.51
~~  54          12  7  3  6  0  0  0  0  0  0                      -19.15~~
    55          12  8  2  3  0  0  0  0  0  0                      -18.97
    56          12  9  1  3  0  0  0  0  0  0                      -18.87
    57          12  1  2  3  6  0  0  0  0  0                      -19.09
~~  58          11  7  5  6  0  0  0  0  0  0                      -18.97~~
    59          11  8  2  5  0  0  0  0  0  0                      -18.79
    60          11  9  1  5  0  0  0  0  0  0                      -18.69
    61          11  1  2  5  6  0  0  0  0  0                      -19.51
~~  62          10  7  4  6  0  0  0  0  0  0                      -19.05~~
    63          10  8  2  4  0  0  0  0  0  0                      -18.88
    64          10  9  1  4  0  0  0  0  0  0                      -18.78
    65          10  1  2  4  6  0  0  0  0  0                      -19.60
    66           9  1  3  4  5  0  0  0  0  0                      -19.65
    67           8  2  3  4  5  0  0  0  0  0                      -19.74
~~  68           7  3  4  5  6  0  0  0  0  0                      -19.92~~
    69           1  2  3  4  5  6  0  0  0  0                      -20.46
```

Figure III-7. Table of Net Benefits of Combinations of Watersheds-No Reuse
Computer Run A

D. Optimization of System

The discussion of sizing criteria for multiple purpose reservoirs which appear in Section II-F-3 included reference to various physical and aesthetic constraints which exist at reservoir sites. Since the application of these criteria is a function of the size of reservoir required at a given site as well as the type of reservoir selected, application of these constraints was not attempted within the systems analysis. In order to define the final optimum system, however, tentative optima must be tested for the physical and aesthetic constraints not included in the computer analysis. Many of these factors are determined by the exercise of judgement on the part of the engineer, the owner, and the owner's representatives, including architects and land planners. For these reasons, and others, they are not considered susceptible to quantitative analysis. The method used to test for these constraints was as follows:

(1) The total net benefits for a particular section of the watershed and particular level of output were placed in order of decreasing net benefit;

(2) The highest-ranking system was defined in terms of reservoir types, sizes, and locations;

(3) Each location was examined physically and via topographic and land use maps for feasibility of installation of the required facility;

(4) If any of the reservoirs in a given combination appeared not feasible due to insufficient usable space, appearance, impact on the neighborhood, etc., that combination was discarded and the next highest-ranking one selected;

(5) The process was repeated until the optimum feasible system was located for the section and output level;

(6) The same method was applied to each output level and to each watershed section until all final optimum systems were defined.

These final systems are described and discussed in Chapter IV. Information and guidance was obtained from the planning and engineering staff of the Howard Research and Development Corporation, developers of Columbia, with respect to land uses, aesthetic considerations and acceptability. The responsibility for the exercise of judgement in qualitative matters rests, however, solely with the staff of Hittman Associates, Inc., who performed the analysis and made the final decisions.

It is interesting to note that the system actually selected from computer run A, all-reuse-level option, was the thirty-first ranking system. Its total net benefit was $20.79 per day lower than the highest-ranking system. Had the local reuse system been a factor in the original planning of this area, it is reasonable to assume that land planning would have provided for a system of significantly higher net benefit, thus improving the economics of the concept over those detailed in this report.

E. Systems Analysis Computer Program

The method employed to analyze and optimize the local reuse system has been described in detail in this and the previous chapter. The actual computations and organization of data referred to in the report were performed on an IBM 360-65 digital computer system, utilizing a computer code developed as part of this study. Appendix C is the complete flow chart of that computer code, showing the actual operation of the various sub-routines and the main program.

IV. LOCAL STORAGE, TREATMENT, AND REUSE SYSTEM CONCEPTUAL DESIGN

A. Approach

The conceptual design and cost estimates of the local storage, treatment and reuse system were prepared for three cases. These are:

(1) Local Collection, Storage, and Treatment of Storm Water for Potable Reuse

(2) Local Collection, Storage, and Treatment of Storm Water for Sub-Potable Reuse

(3) Local Collection, Storage, and Treatment of Storm Water for Pollution Control

The system analysis and optimization studies, discussed in the two previous chapters had provided a complete analysis of the various combinations of components and locations based on least overall cost. The system analysis program had not been programmed to evaluate actual site constraints and other design conditions. Accordingly, the development of conceptual designs and cost data involved (1) the selection of the system configuration and locations which could be constructed and operated in the Wilde Lake Watershed at the least overall cost, (2) the preparation of typical designs and drawings, (3) modification of costs based on actual site conditions, and (4) compilation of performance characteristics and cost data.

1. Selection of Plant Locations and System Configurations

A typical output of the system optimization program was previously shown as Figure III-6, "Table of Net Benefits for Combinations of Watersheds" and was discussed in Section III.C. Similar output sheets were produced for each of the three sectors of the Wilde Lake Watershed for which conceptual designs were desired. These tables provide a listing of the various combinations of plants and the net benefit (or cost) expressed in terms of dollars per day.

As a first step it was necessary to examine the topography, boundaries, and other site conditions to determine the maximum size reservoir that could be constructed at each possible location. This involved plotting the contours of the maximum water levels associated with various heights of dams, planimetering storage areas, estimating additional storage quantities that could be obtained by excavation, etc. In this examination, the existing land use plans and lot boundaries were considered invariant to assure a practical design based on actual site conditions.

The next step, was to determine the available storage capacities of various combinations of reservoirs and to compare the available storage with the calculated volume for storms of a one-year return interval. For each type of local storage system, the plant combination showing the maximum benefit or least cost was found from among the first feasible plant combination in the case of systems for potable reuse for the southwest section of the Wilde Lake Watershed, (Combination Number 46, Figure III-6).

Having selected the least cost and first feasible plant combination, the system configuration was also identified. The computerized system analysis program automatically calculates the capital and operating costs of alternative systems and selects the least cost configuration. Accordingly, it was only necessary to examine the printout of the program to determine the components that had been selected by the computer program for the various plants. In all cases, the least cost plant was based on the use of open storage reservoirs. The system analysis program, also calculates and prints the amount of water supplied for reuse.

2. Preparation of Plant Location and Typical Design Drawings

The conceptual design drawings showing the plant locations were prepared based on the selected least cost plant combinations. In the case of the "Local Collection Storage and Treatment of Storm Water for Reuse," it was also necessary to layout a distribution system. These drawings are contained in Sections IV. B, C, and D with a discussion of the conceptual designs of the three systems.

Since this system study also involved the preparation of conceptual design drawings of the facilities for a demonstration program, the locations for the demonstration facilities were used as the basis for the preparation of typical design drawings for storage reservoirs, pretreatment systems and treatment plants. These drawings are contained in Chapter VII and are further described in Appendix F.

3. Modification of Costs Based on Actual Site Conditions

The typical design drawings of the facilities for the demonstration program were also used for another purpose. Detailed unit and material takeoffs were made and cost estimates based on the actual designs were prepared. These cost estimates were compared to the capital costs generated by the system analysis program. The capital cost inputs for the system analysis program had been based on parametric cost data as a function of reservoir capacity and treatment rates. Although all the major components were considered, it was not possible to consider all of the possible site conditions in this type of analysis. Based on the comparison of the computer generated costs and the estimates based on the typical designs, prepared for the demonstration program, the cost data used for the storage reservoirs, and pretreatment units were found to be low. This was due to the following local conditions:

(a) The storage reservoirs would be "Class 3 Impoundments" in accordance with recent regulations of the State of Maryland. "Class 3 Impoundments" are required to have spillways designed on the basis of a 100 year storm interval intensity. This regulation requires a dam of greater height and longer wing walls to retain the water. Approximately two feet was added to typical embankment height and embankment lengths were generally doubled.

(b) The reservoirs for the demonstration programs were designed to have a minimum pool depth of two feet to preclude weed growth. This, plus the "Class 3 Impoundment" requirement, increases the amount of excavation required.

(c) The demonstration plant estimate includes fencing around the reservoirs.

(d) The allowances for contingencies and engineering in the demonstration plant are based on "first of a kind" facilities and the construction of a limited number of facilities.

The treatment plant designed for the demonstration program would probably not be typical of those used for the treatment of storm water in future installations, particularily where the quantity of water is relatively small. This plant is designed as a general purpose test plant in which it would be possible to evaluate various treatment processes. Once treatment processes are established for storm water, it is most likely that packaged treatment plants would be designed for this purpose. Appendix H contains information on the plants of this type. These could either be installed in underground chambers or housed in small surface structures.

For purposes of estimating the capital costs of constructing local storage, treatment and reuse systems in the Wilde Lake Watershed, the estimated costs for the demonstration program storage reservoirs and treatment units were used in order to be conservative. This was done by plotting the estimates for the three demonstration reservoirs and pretreatment systems on the parametric cost curves used in the system analysis. New curves of costs vs. capacity were generated and the appropriate costs were determined for each of the reservoirs required. Treatment plant costs were not modified. The treatment cost data used in the system analysis program is considered conservative, particularly since packaged treatment plants such as those described in Appendix H would be used.

4. Compilation of Performance Characteristics and Cost Data

Each of the three local collection, storage, and treatment, systems consists of a number of reservoirs, pretreatment, treatment, and distribution facilities. Accordingly, the conceptual design of the system involved the compilation of the performance data for each of the individual installations and the evaluation of the overall effectiveness as a means of pollution control and water supply augmentation for the entire watershed. Similarly the capital and operating costs of the individual installation were compiled to evaluate the overall cost of the system, the economic benefits of reuse and the net costs of pollution control. These costs were then broken down to establish the costs per acre, per dwelling unit, and per thousand gallons of runoff. The performance and cost data for each of the three local collection, storage and treatment systems is presented in the next three sections of this Chapter.

B. Local Collection, Storage, and Treatment of Storm Water for Potable Reuse

1. System Description

The "Local Collection, Storage, and Treatment of Storm Water for Potable Reuse" system, hereafter referred to as the "Potable Reuse System" consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated sub-watersheds and treated water and overflows from the upstream storage reservoirs. Table IV-1, lists the locations of the ten reservoirs and the sub-watersheds and upstream reservoirs from which flow is collected. The locations are shown on Figure IV-1.

TABLE IV-1. POTABLE REUSE SYSTEM STORAGE RESERVOIR COLLECTION AREAS

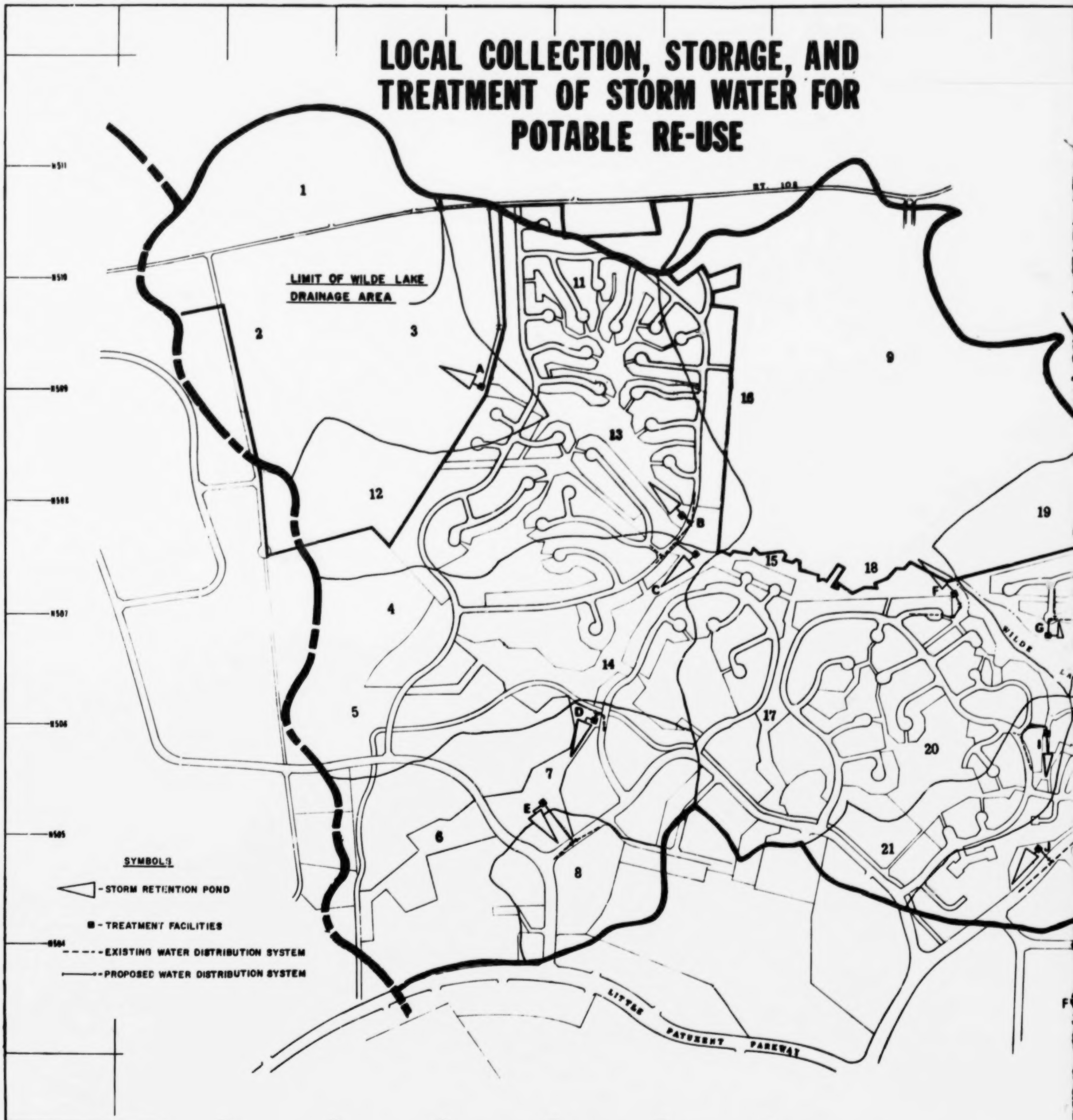
	Source of Flow						
Location	Sub-Watershed						Upstream Reservoir
A	1	2	3				-
B	11	12	13				A
C	4	5	14				D
D	6	7					E
E	8						-
F	9	15	16	17	18	20	B, C
G	19						-
H	22						-
I	21						J
J	10*						

*The only feasible location for a storage reservoir in sub-watershed 10 is Location J. At this location only 40 percent of the flow from the sub-watershed is intercepted.

A pretreatment unit is installed at each of the storage reservoirs. These pretreatment units, treat the storm water to Class "C" quality for discharge to Wilde Lake or for further treatment and reuse. Treatment plants are located at nine of the storage reservoirs. The treatment plants take Class "C" water from the pretreatment units and treat the water to potable quality, Class "AA." Table IV-2 lists the type of storage, storage capacity, pretreatment capacity, quality of water produced, and the final treatment plant capacity. No treatment plant is provided at Location H, because incremental costs of treatment exceeded possible benefits from reuse. Pretreatment units are sized for maximum flow independent of water diverted for treatment upstream.

Each of the nine treatment plants includes pumping facilities and connections to the public water systems. The treated storm water is supplied as a supplementary source of water and is distributed through the public water distribution systems to meet all types of demands, Classes "AA", "A", "B", and "C."

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR POTABLE RE-USE


RT. 108
LIMIT OF WILDE LAKE DRAINAGE AREA
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B
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D
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I
J
WILDE LAKE
LITTLE PATUXENT PARKWAY
N511
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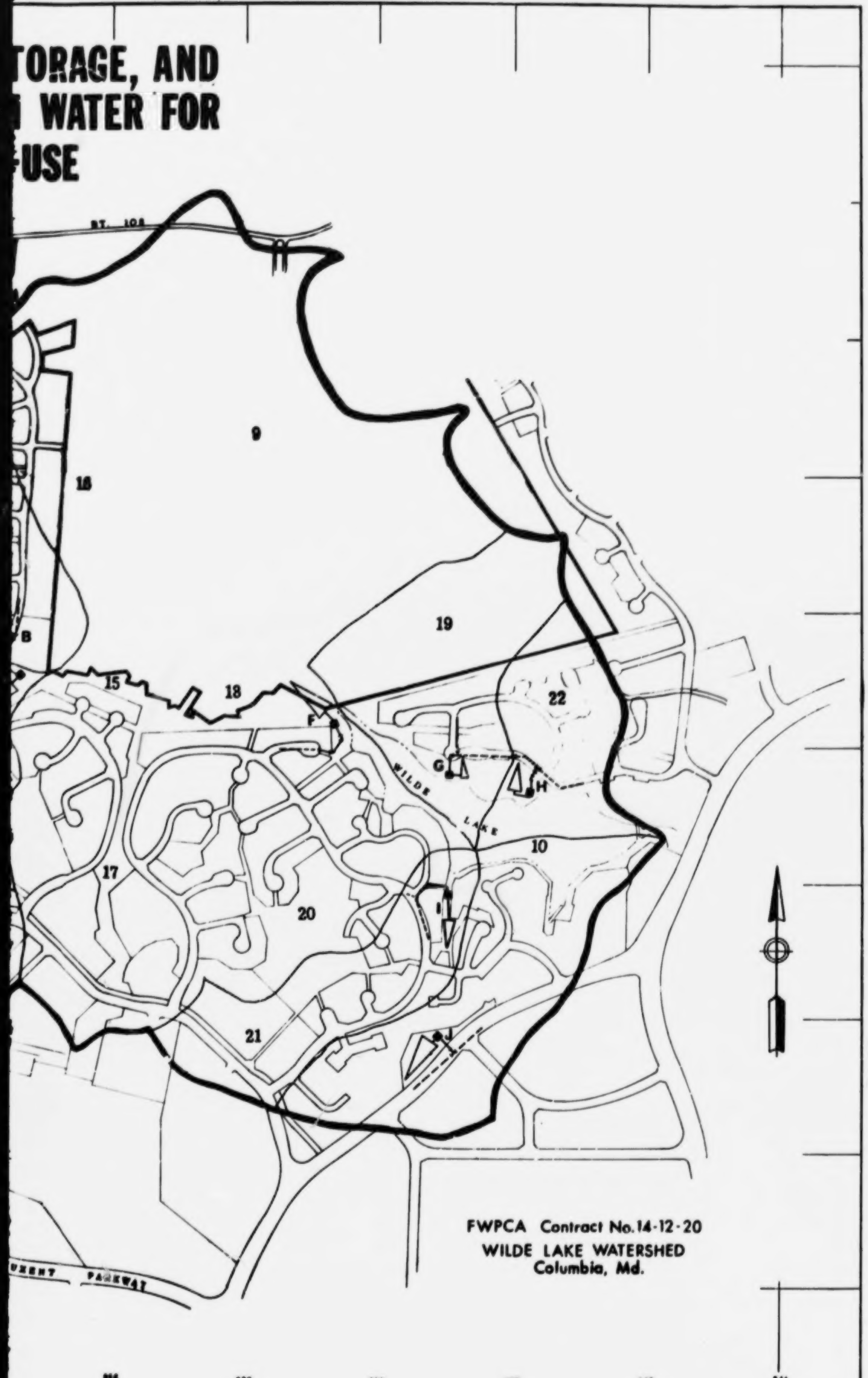
N504
831
832
833
834
SYMBOLS
- STORM RETENTION POND
- TREATMENT FACILITIES
- EXISTING WATER DISTRIBUTION SYSTEM
- PROPOSED WATER DISTRIBUTION SYSTEM
Hittman Associates, Inc.
COLUMBIA, MARYLAND

A


TORAGE, AND
WATER FOR
USE
RT. 108
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FWPCA Contract No.14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
UXENT PARKWAY

Figure IV-1. Potable Reuse System Location Plan

B

TABLE IV-2. LOCAL, COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR POTABLE REUSE - CONCEPTUAL DESIGN

Location	Type of Storage	Capacity (Gal)	Pretreatment Capacity (gpd)	Water Quality Produced	Final Treatment Capacity (gpd)	Connected Demands
A	POND	1,577,300	67,900	AA	30,400	AA, A, B, C
B	POND	2,145,000	231,400	AA	81,800	AA, A, B, C
C	POND	1,902,100	386,100	AA	76,900	AA, A, B, C
D	POND	1,907,200	232,400	AA	86,800	AA, A, B, C
E	POND	625,900	58,900	AA	29,500	AA, A, B, C
F	POND	5,101,400	1,036,300	AA	209,400	AA, A, B, C
G	POND	923,500	85,700	AA	38,400	AA, A, B, C
H	POND	358,400	29,800	C		
I	POND	690,000	63,800	AA	31,900	AA, A, B, C
J	POND	503,200	55,100	AA	24,780*	AA, A, B, C

*40 percent of watershed intercepted by storage facility.

2. Component Descriptions

Figure IV-2 is a flow diagram of a typical "Potable Reuse System" installation. Each of the installations (except at Location H) consists of the following:

a. Storage Reservoir. These reservoirs are open storage reservoirs, or ponds, formed by the construction of concrete dams. Typical storage reservoirs are shown in Figure 2 and 3 of Appendix F.

b. Pretreatment Units. The pretreatment units are installed downstream and receive flow by gravity from the storm retention reservoirs. As shown in Figure 2 of Appendix F, the pretreatment units consist of a float-operated valve, chlorination equipment chamber, and a tube settler. The pretreatment units have a sump for the collection of sediment and use float valves to control flow.

c. Treatment Plant. The treatment plant takes flow from the pretreatment unit. The plant consists of a mixed media filtration unit followed by an activated carbon column for carbon absorbtion. Provision is made for chemical feed prior to filtration and for chlorination prior to discharge to the treated water storage tank. The piping is arranged for backwashing the mixed media filter with a connection to the sanitary sewer.

d. Treated Water Storage. The treated water storage consists of steel tanks at grade. The treated water storage tank also serves as the filter backwash storage tank, the distribution pump clearwell, and the chlorine contact tank.

e. Pumping System. The pumping distribution system consists of a pump with associated control and a connection for supplying treated water to the public water system.

f. Sewer Connection. A connection is provided to public sewer systems to receive filter backwash. Appropriate measures are taken to prevent any possibility of backflow from the sewer.

3. Performance Characteristics

The demands for potable water in the Wilde Lake watershed will exceed the amount of storm water runoff available even with the higher runoff quantities that will result when the area is further developed. In addition, since the treated storm water would be supplied to and distributed by the public water system, the demands of any part of the watershed can be supplied. This is contrasted to systems using a separate distribution system in which the supply and demand of the individual subwatershed must be considered. Table IV-3 contains a listing of the annual potable water requirements, the total amount of storm water runoff, and the amount of treated water supplied to the service areas from the treatment plants.

Since the demands of the watershed exceed the available supply, the "Potable Reuse System" was designed to produce the maximum treated water in order to maximize the reuse benefit. An upper limit of 90 percent of the available storm runoff was used in the design of the individual

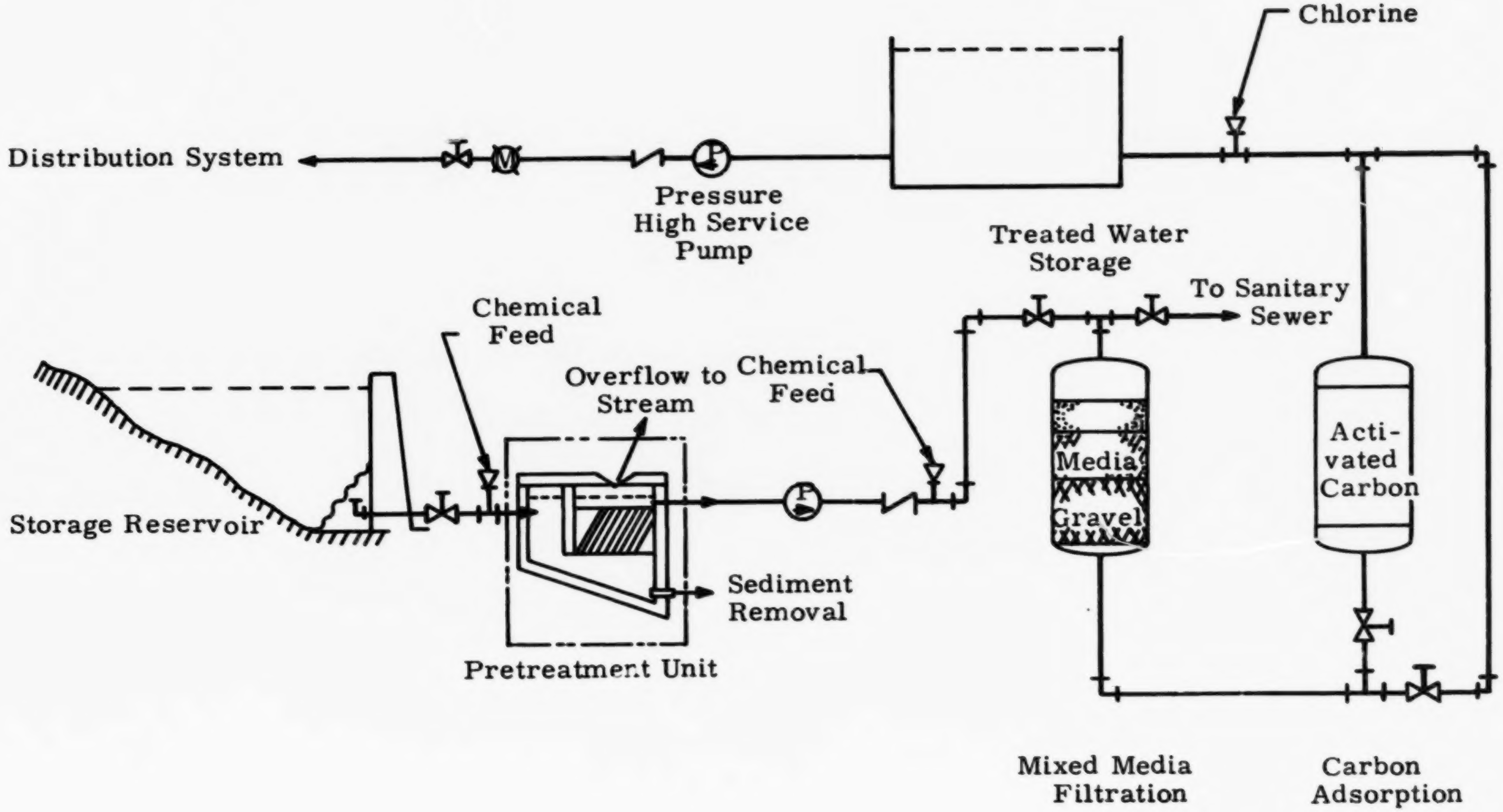

Chlorine
Distribution System
Pressure High Service Pump
Treated Water Storage
To Sanitary Sewer
Chemical Feed
Overflow to Stream
Chemical Feed
Media
Gravel
Acti-vated Carbon
Storage Reservoir
Sediment Removal
Pretreatment Unit
Mixed Media Filtration
Carbon Adsorption

Figure IV-2. Potable Reuse System Flow Diagram

TABLE IV-3. POTABLE WATER REQUIREMENTS, TOTAL STORM WATER RUNOFF, AND AMOUNT SUPPLIED
(Amounts in Gallons/Year)

		Potable Water Requirement		Total Storm Water Runoff		
Treatment Plant Location	Sub-watersheds	By Sub-watershed	Pretreatment Plant	By Watersheds	Per Treatment Plant	Amount Supplied
A	1	4,519,000		3,627,000		
	2	4,551,000		4,694,000		
	3	1,858,000	10,928,000	4,073,000	12,394,000	11,104,500
B	11	15,283,000		12,378,000		
	12	7,234,000		7,332,000		
	13	11,852,550	34,369,000	10,134,000	29,844,000	26,754,500
C	4	9,278,000		4,791,000		
	5	10,289,000		7,733,000		
	14	24,229,000	43,796,000	15,528,000	28,052,000	25,155,800
D	6	67,435,000		17,653,000		
	7	35,679,000	103,114,000	14,007,000	31,660,000	28,185,300
E	8	16,542,000	16,542,000	10,741,000	10,741,000	9,621,400
F	9	22,907,000		17,818,000		
	15	3,927,000		3,700,000		
	16	15,830,000		10,260,000		
	17	24,794,000		20,388,000		
	18	6,216,000		3,905,000		
	20	20,122,000	93,796,000	20,351,000	76,422,000	68,612,700
G	19	8,399,000	8,399,000	15,637,000	15,637,000	14,009,900
H	22	7,296,000	7,296,000	5,440,000	5,440,000	
I	21	22,601,000	22,601,000	11,653,000	11,653,000	10,453,600
J	10	45,045,000	45,045,000	25,124,000	10,049,000*	9,044,500
Totals		385,886,000	385,886,000	246,967,000	231,892,000	202,942,200

*Intercepts 40 percent of the sub-watershed.

reservoirs and treatment plants, in order to assure that a portion of the storm runoff would still go to Wilde Lake, in addition to the base flow. As finally designed the "Potable Reuse System" will provide 202,942,000 gallons per year of calculated annual runoff of 246,967,000 gallons. This represents 82 percent of the available supply. About 29,000,000 gallons of treated water would be released to Wilde Lake and an additional 15,000,000 gallons of storm water runoff would go directly to the Lake since the only possible location for a reservoir in Sub-watershed 10 is at Location J which can only intercept some 40 percent of the runoff from the sub-watershed.

The pretreatment units are designed to treat storm water at twice the mean annual daily runoff rate, $2 \times \bar{q}$. In order to maximize the treated water supply the pretreatment units would be operated at 1.1 to $1.4 \times \bar{q}$, plus the base flow. At this rate, there is an increased probability of not having the reservoir emptied sufficiently to receive the runoff from a subsequent storm. In terms of the design criteria of storms of one year reoccurance intervals, the $2 \times \bar{q}$ discharge rate would retain 98.5 percent of the runoff whereas a $1.13 \times \bar{q}$ discharge rate would retain 97.3 percent of the flow. Expressed in terms of all storms including those greater than one-year recurrance interval, the following relationships were calculated.

Condition	Percent of Total Runoff Retained
One year storm design - no residual assumed	92.6
One year storm design - $2 \times \bar{q}$ discharge rate	91.2
One year storm design - $1.13\ \bar{q}$ discharge rate	90.0

Accordingly, with the "Potable Reuse System" not more than 10 percent of the runoff would normally overflow the reservoir and not receive the full Class "C" treatment. However, even the overflows will be retained in the storage reservoirs for a time period adequate to allow settlement of the heavier particulate matter. Added to the runoff that cannot be practically intercepted this would represent 22,000,000 to 40,000,000 gallons per year compared to the 247,000,000 gallons or greater amount of runoff that would otherwise be discharged directly to the lake. Neither estimate includes the base flow from the watershed which is not affected by individual storms.

Most of the water reused by the "Potable Reuse System" would eventually return to the river basin as effluent from the wastewater treatment plant. Since this is downstream from Wilde Lake, the remaining flow to the Lake must also be considered. Water must be supplied to Wilde Lake to provide for the evaporative losses, seepage from the lake and the release requirements placed on the Wilde Lake dam by the State of Maryland. The evaporative losses of the lake were calculated to be about 22,000,000 gallons per year. However, the rainfall directly into the lake represents an equal or slightly greater amount of water. The State requirement on release is a general requirement that precludes diversion of the water from the Lake itself.

The 29,000,000 gallons of treated water and the 22,000,000 to 40,000,000 gallons of untreated water that would be discharged to the lake did not include the base flow from the stream. As discussed in Section II.C, a temporary gaging station was installed and operated on a 130 acre sub-watershed in Wilde Lake as part of this study. The measured base flow of 0.05 cfs, extropolated to the entire 1,140 acre watershed, would indicate an annual base flow to the lake in excess of 104,000,000 gallons per year. The total flow of 150 to 170 million gallons of water per year to Wilde Lake would correspond quite closely to the natural condition of the area with a runoff calculated at 43,000,000 gallons per year plus the base flow.

The 202,942,000 gallons per year of treated water supplied to public water system represents 52.6 percent of the total demand of the watershed. At the current distributed price of 45 cents per thousand gallons, this represents an annual benefit of $91,324.

4. Design and Construction Costs

The "Potable Reuse System" will consist of 10 storage reservoirs and pretreatment units and nine storm water treatment plants, pumping systems and connections to the public water and sewer system. The locations, sizes, and treatment rates of these facilities are shown in Figure IV-1 and Table IV-2 and are further described in Section IV.B.2. The major design and construction items are as follows:

a. Storage Reservoirs
    - Site Work
    - Excavation
    - Embankment
    - Concrete Spillway and Walls
    - Grouted Rip Rap

b. Pretreatment Units
    - Chamber
    - Effluent Piping
    - Chlorination
    - Tube Settler

c. Treatment Plant
    - Excavation
    - Structure
    - Mechanical Equipment
    - Electrical Equipment
    - Transformer and Electrical Distribution

d. Pumping and Distribution
    - Pumps and Controls
    - Treated Water Storage
    - Connections to Public Water System
    - Connection to Public Sewer
    - (Electrical transformers and distribution plan having been included as part of the treatment facility)

e. Design (Estimate at 15 percent of Construction Costs)
    - Drawings and specification Preparation
    - Field Engineering and Inspection
    - Project Management

f. Contingencies and Escalation
    - (Estimated at 10 percent of Construction Costs)

As discussed in Sections II and III, land costs were not included on the basis that all of the storage reservoirs and treatment plants could be located within the flood plain which is dedicated open space in the case of Columbia.

Table IV-4 contains a summary of the estimated design and construction costs of a "Potable Reuse System" for the entire Wilde Lake watershed, by location and component parts.

In order to provide conservative estimates of the design and construction costs for the "Potable Reuse System," for comparison with other systems, the cost estimates in Table IV-4 are based upon the assumptions and criteria used in the system analysis. As noted previously in Section IV. A. 3, the estimate costs of the storage reservoirs and pretreatment units have been scaled up to take full account of local conditions. It is considered that these costs could be reduced substantially by further design work. Likewise, estimates for the treatment plants are based on small size conventional treatment plants. It is envisioned that packaged treatment plants would be developed for future "Potable Reuse Systems" and these costs would also be substantially reduced.

5. Operation and Maintenance Costs

The operation and maintenance costs for the "Potable Reuse System" were estimated using the assumptions and criteria used in the system analysis as discussed in Chapter II. Since a number of nearly identical installations are involved which would be located in a relatively small area, the parametric cost estimates were modified based on the actual designs that had been developed. For example, checks and operations can be performed by personnel moving from one location to another. Likewise, maintenance items such as the cleaning of sediment from reservoirs can be accomplished by a crew handling a number of facilities at one time. Certain of the costs, i.e., chemicals, pumping power, etc., can be directly related to the quantities of water processed. As discussed in Chapter II, the costs for operation and maintenance as based on this work being performed by extension and expansion of an existing organization and a supporting organizational and overhead structure has not been included.

Table IV-5 contains a summary of the operation and maintenance for the "Potable Reuse System" prepared on the following basis:

(a) Storage Reservoirs

(1) Routine checks based on weekly checks requiring one half man-hour per reservoir per check

(2) Trash Removal and Nuisance Control on monthly basis requiring one half man-day per reservoir plus chemicals at $25 per reservoir per year

(3) Sediment Removal on a semi-annual basis requiring a front loader and operator, and two trucks and driver for one half day per reservoir

TABLE IV-4. LOCAL COLLECTION, STORAGE, TREATMENT OF STORM WATER FOR POTABLE REUSE DESIGN AND CONSTRUCTION COSTS

(Amounts in Dollars)

	Location										
	A	B	C	D	E	F	G	H	I	J	Total
Construction											
Storage Reservoirs	61,000	71,500	67,500	68,000	40,500	109,000	47,000	32,500	42,000	33,500	
Pretreatment Units	8,200	9,600	10,200	9,600	8,000	11,400	8,600	7,300	8,200	7,600	
Treatment Plant	35,600	41,800	41,100	42,400	35,400	78,400	36,600		35,800	35,000	
Pumping and Distribution	12,400	12,320	12,600	12,800	10,200	19,400	10,800		10,600	12,100	
Subtotal	117,200	135,200	131,400	132,800	94,100	218,200	103,000	39,800	96,600	87,600	1,155,900
Design, Inspection and Field Engineering	17,500	20,300	19,700	20,000	14,100	32,700	15,600	6,000	14,600	13,200	173,700
Contingencies and Escalation	11,700	13,500	13,100	13,300	9,400	21,800	10,300	4,000	9,700	8,800	115,600
Total	146,400	169,000	164,200	166,100	117,600	272,700	128,900	49,800	120,900	109,600	1,445,200

TABLE IV-5. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR POTABLE REUSE - ANNUAL OPERATING AND MAINTENANCE COSTS

	Labor (man hours)	Labor ($)	Material ($)	Equip. Rental* ($)	Miscellaneous ($)	Totals
Storage Reservoirs						4,090
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690	250			
(3) Sediment Removal				2,400		
Pretreatment Units						
(1) Routine Checks	260	750				
(2) Sediment Removal				3,250		10,110
(3) Chemicals			4,500			
(4) Preventative Maintenance and Repair	610	1,760	880			
Treatment Plants						35,030
(1) Routine Inspections	2,920	8,400				
(2) Water Tests	940	3,060	400			
(3) Routine Operation	1,880	5,400				
(4) Chemicals			2,500			
(5) Preventative Maintenance	2,350	7,640	3,820			
(6) Electrical Energy					3,000	
(7) Sewer Change					810	
Pumping Facility						
(1) Routine Checks	240	715				6,390
(2) Preventative Maintenance and Repairs	700	2,260	1,130			
(3) Electrical Energy					3,000	
(4) Distribution		120				
Miscellaneous			14,090			14,090
Totals	10,436	31,545	26,840	5,650	6,810	70,845

*Include Operators

(b) Pretreatment Units

  (1) Routine Checks based on weekly checks requiring one half man-hour per reservoir

  (2) Sediment Removal - based on removal of 750 gallons of sludge having 10 percent solids on a biweekly basis using a tank truck with pump, and requiring a driver and helper for two hours per reservoir

  (3) Chemicals based on 85 pounds of alum and 4.25 pounds of calcium hyprochlorite per million gallons of runoff

  (4) Preventive Maintenance and Repair - based on a materials cost of one half percent per year and labor at twice materials cost

(c) Treatment Plants

  (1) Routine Inspection - based on daily inspection requiring one man-day for all plants

  (2) Water Quality Tests taken as part of routine inspections and requiring an average of 2 man-hours per week per treatment plant for analysis plus chemicals and supplies at $400

  (3) Routine Operation, backwash of filters, adjustment of equipment, and replenishment of filters, are based on one-half man day per week per plant in addition to routine inspection

  (4) Chemicals based on 50 pounds alum and 8.5 pounds calcium hyprochlorite per million gallons of treated water

  (5) Preventive Maintenance and Repairs based on materials at one percent of the capital cost per year per plant with labor at twice materials

  (6) Electrical Energy based on the equivalent of 2 pumps operating full time per plant

  (7) Sewer charge based on one percent of the treated flow at 42 cents per 1,000 gallons

(d) Pumping Facility

  (1) Routine checks performed as part of treatment plant inspections

  (2) Preventive Maintenance and Repair - material cost based on one percent of capital cost with labor twice material costs plus $0.05 per square foot tank painting per year

  (3) Electrical Energy based on one pump operating at 80 psi head continuously

  (4) Distribution System maintenance and repair at 0.6 percent of capital cost annually

(e) Miscellaneous

(1) Other direct costs at 10 percent

(2) Indirect costs of 10 percent

(3) Travel at $2.00 per man day

Labor is estimated at $2.88 per hour for semi-skilled labor and $3.25 per hour for technicians and maintenance.

6. Sediment Disposal

In this study, it was assumed that a suitable disposal site would exist within a reasonable distance from the storage reservoirs and that the accumulated sediment from the reservoirs and pretreatment units would be hauled to the site and dumped. This is admittedly a gross simplification since the disposal of the sediment can represent a major problem during the period that an area is undergoing urbanization. A detailed study of sediment disposal methods was beyond the scope of this study. Since the problems of sediment disposal would be comparible for both the local storage systems and the conventional pollution control system, the incremental costs for disposal would be essentially the same, and the overall conclusions of the study would not be altered. As discussed later in Section VI-E, Secondary Benefits, the local pollution control systems could reduce the overall quantities of sediment to be handled through the abatement of post development stream erosion.

As noted above, the major portions of the sediment would come from the erosion during construction. After the area is developed and planted, the quantities and frequency of removal would be substantially reduced. If a suitable disposal site for the dumping of the sediment does not exist, it would be necessary to construct facilities for drying of the sediment. These would probably be sand drying beds designed to permit the use of mechanical equipment for dumping and removal of the sediment. Since the quantities of material to be handled after the area becomes developed will be quite small, the major portion of such a dumping facility could be temporary. After drying, it is considered that the material might be blended with other material for backfill or perhaps restored for use as top soil. Another alternative would be to use the material as cover for sanitary landfill. Over the overall life of a pollution control system, the incremental costs for the drying and disposal of sediment would not be a major portion of the total costs.

C. Local Collection, Storage and Treatment of Storm Water for Sub-Potable Reuse

1. System Description

The "Local Collection, Storage, and Treatment of Storm Water for Sub-Potable Reuse" system, hereafter referred to as the "Sub-Potable Reuse System, " consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated sub-watersheds and treated water from the upstream storage reservoirs. Table IV-6 lists the locations of the ten storage reservoirs and the sub-watersheds and upstream reservoirs from which flow is collected. The locations are shown on Figure IV-3.

TABLE IV-6. SUB-POTABLE REUSE SYSTEM STORAGE RESERVOIR COLLECTION AREAS

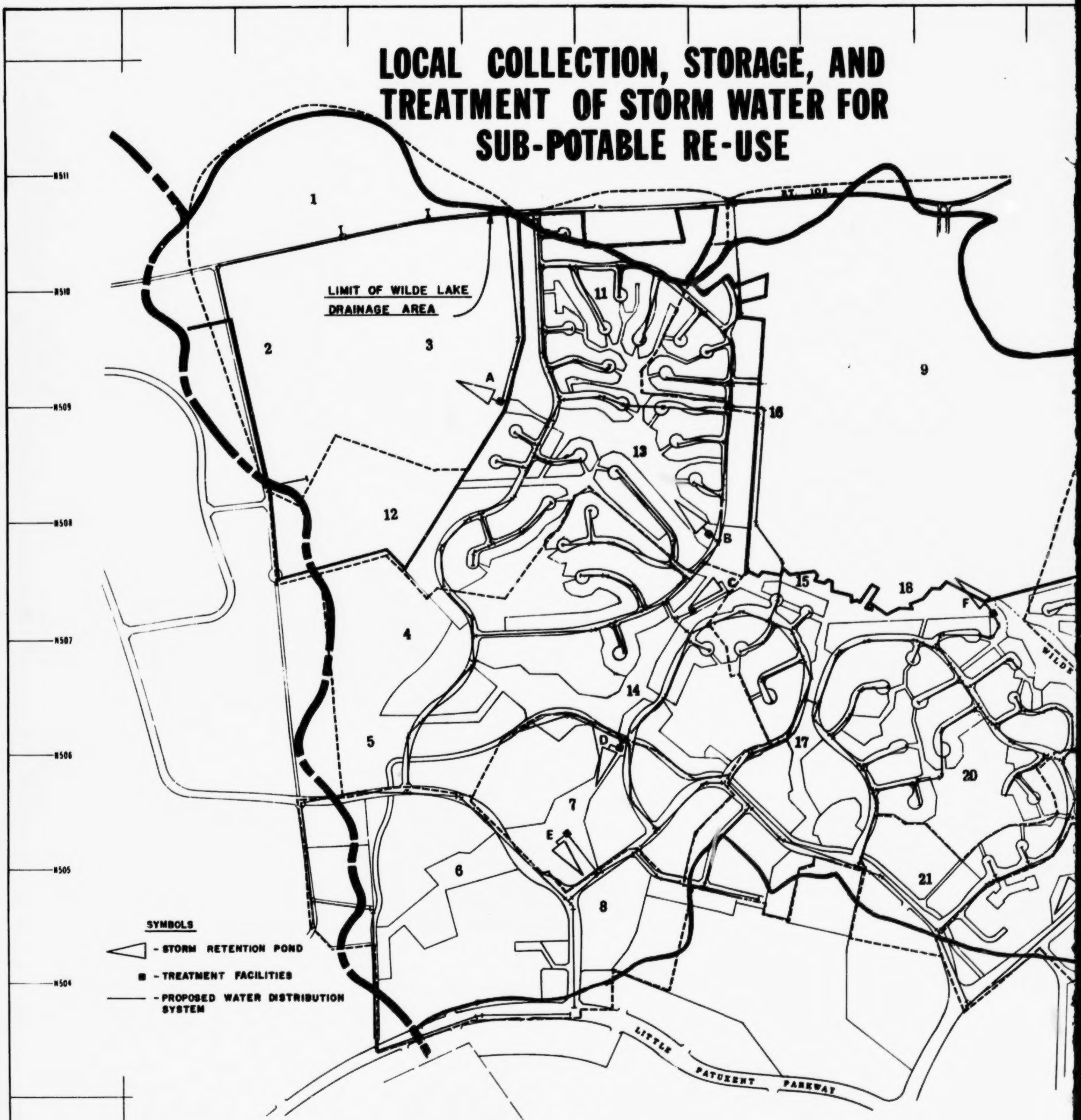
	Source of Flows					
Location	Sub-watersheds					Upstream Reservoirs
A	1	2	3			-
B	11	12	13			A
C	4	5	14			D
D	6	8				E
E	7					-
F	9	15	16	17	18 20	B, C
G	19					
H	22					
I	21					
J	10*					

*Reservoir J only intercepts 40 percent of sub-watershed 10.

The "Sub-Potable Reuse System, " like the "Potable Reuse System, " has pretreatment units which treat the storm water to Class "C" quality for discharge to Wilde Lake or for further treatment. In the case of the "Sub-Potable Reuse System" the water from the pretreatment units is either distributed as Class "B" or Class "C" water. In the case of Class "B" reuse, a treatment plant is installed with a pumping station which takes the water from Class "C" to Class "B" quality. In the case of Class "C" reuse, the water is taken from the pretreatment units through a pumping station. As shown on Figure IV-3, a distribution system is provided for the sub-potable water in each sub-watershed. Connections are provided from the pumping stations to the distribution system in the sub-watershed or combination of sub-watersheds from which runoff is collected.

Table IV-7 lists the type of storage, storage capacity, pretreatment capacity, water quality produced, final treatment quality, and the

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR SUB-POTABLE RE-USE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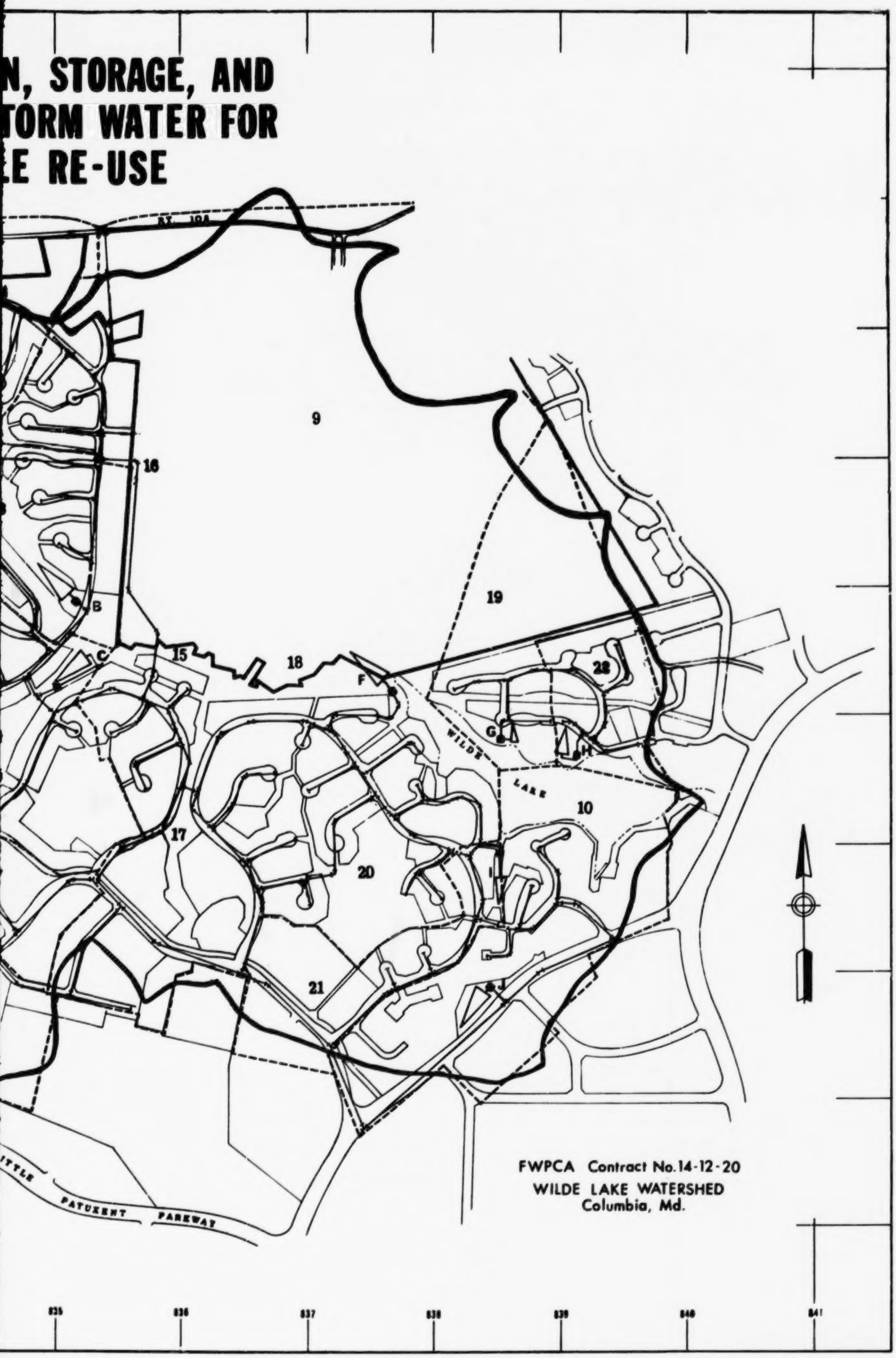
SYMBOLS
- STORM RETENTION POND
- TREATMENT FACILITIES
- PROPOSED WATER DISTRIBUTION SYSTEM
Hittman Associates, Inc.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PATUXENT PARKWAY
FWPCA Contract No.14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
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841

B

Figure IV-3. Sub-Potable Reuse System - Location Plan

TABLE IV-7. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR SUB-POTABLE REUSE - CONCEPTUAL DESIGN

Location	Type of Storage	Storage Capacity (Gal)	Pretreatment Capacity (gpd)	Water Quality Produced	Final Treatment Capacity (gpd)	Connected Demands
A	POND	1,577,300	67,900	B	23,500	B, C
B	POND	2,145,000	231,400	B	54,000	B, C
C	POND	1,902,100	386,100	B	65,400	B, C
D	POND	803,000	232,400	C	33,600	C
E	POND	1,729,900	155,600	C	77,800	C
F	POND	5,101,400	1,036,300	B	160,200	B, C
G	POND	923,500	85,700	C	4,300	C
H	POND	358,400	29,800	C	5,400	C
I	POND	690,000	63,800	C	20,100	C
J	POND	503,000*	55,100	C	20,700	C

*40% of watershed intercepted by storage facility connected demands for each of the reservoirs and treatment plants used with the "Sub-Potable Reuse System."

Comparing Table IV-1 and Table IV-6, it is noted that "Potable Reuse System" and the "Sub-Potable Reuse System" differ with respect to the collection areas of reservoirs D and E. As shown on Figure III-6, the optimal feasible case for the "Potable Reuse System" is the Combination Number 46 (Cases 14, 11, and 5) whereas the optimal sub-potable combination is number 48 (Case 14, 10, and 4) for Class "C" water.

2. Component Descriptions

The components comprising the "Sub-Potable Reuse System" installations are similiar to those of the "Potable Reuse System" described in Section IV.B.2. The two types of systems differ only with respect to the treatment plants and the sub-potable distribution system.

The "Sub-Potable Reuse System" has two types of treatment facilities. Figure IV-4 shows a flow diagram for a Class "B" treatment plant of the type used at Locations A, B, C, and F. For this type plant, water from the pretreatment unit goes through a microstrainer and is pumped to a treated water storage tank which also serves as a chlorine contact tank. A high pressure service pump supplies the sub-potable distribution system. A connection from the public water supply system provides water when the local treatment plant is not operating.

Figure IV-5 is a flow diagram for a Class "C" treatment plant of the type used at Locations D, E, G, H, I, and J. With this type plant, flow from the pretreatment unit goes directly to a treated water storage tank which is also a chlorine contact tank and is then pumped to the sub-potable distribution system. A connection from the public water supply is also provided to provide water when the plant is not operating.

The sub-potable distribution consists of underground pipe lines as shown on Figure IV-3 and the separate piping systems within the houses, townhouses, apartments, schools, and commercial buildings. The design is based on the construction of the sub-potable distribution system following installation of other utilities. In actual practice, such a system might be placed at the same time as the public water supply and sewer system. This would appreciably reduce the cost of the sub-potable system. This point is also discussed in Section IV. C.4.

As in the case of the "Potable Reuse System," the physical facilities for the "Sub-Potable System" would be similiar to those for the demonstration facilities, for which drawings are included in Appendix F.

3. Performance Characteristics

In the case of the "Sub-Potable Reuse System" the water is distributed for use to the sub-watersheds from which it is collected. Unlike the "Potable Reuse System," the "Sub-Potable Reuse System" must consider the available supply of water and the demands for sub-potable water on a sub-watershed by sub-watershed basis. Table IV-8 contains a listing of the sub-potable water requirements by class of water to be distributed, for each sub-watershed and treatment plant the total storm water runoff available from each sub-watershed at each treatment plant and the amount of water supplied.

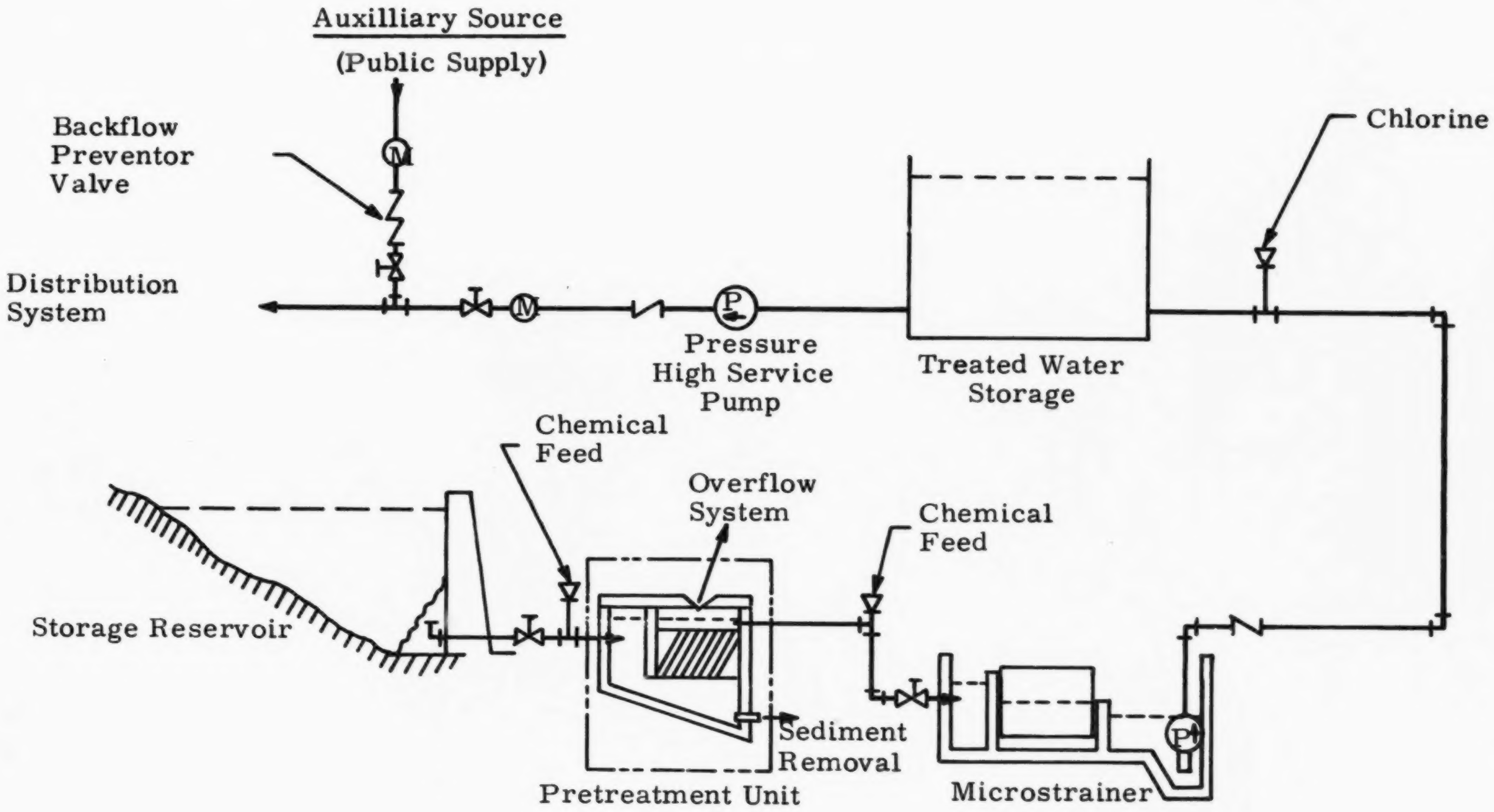

Auxilliary Source
(Public Supply)
Backflow Preventor Valve
Distribution System
Pressure High Service Pump
Treated Water Storage
Chlorine
Chemical Feed
Overflow System
Chemical Feed
Storage Reservoir
Sediment Removal
Pretreatment Unit
Microstrainer

Figure IV-4. Sub-Potable Reuse System Flow Diagram - Class "B" Treatment

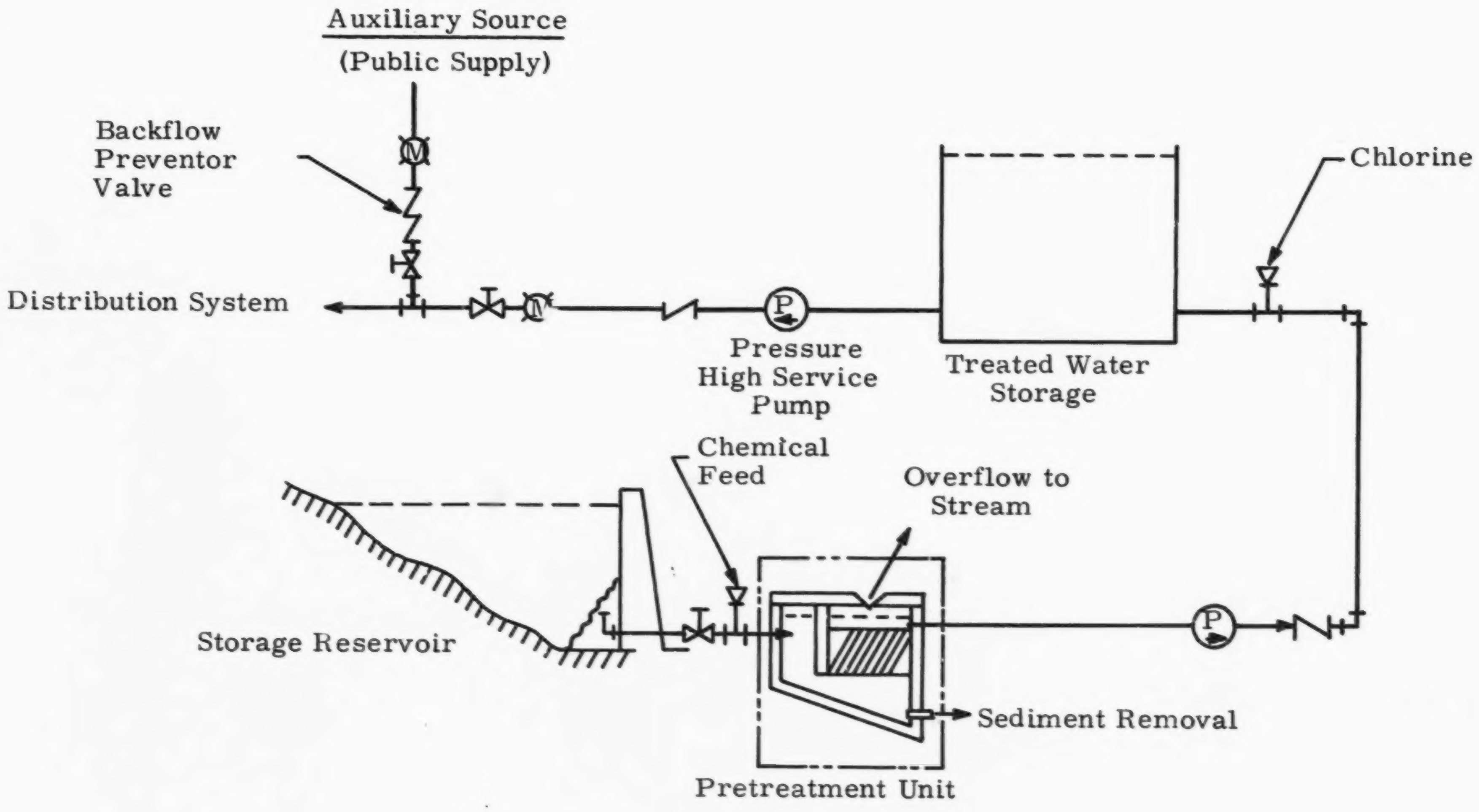

Auxiliary Source
(Public Supply)
Backflow Preventor Valve
Distribution System
Pressure High Service Pump
Treated Water Storage
Chlorine
Chemical Feed
Overflow to Stream
Storage Reservoir
Sediment Removal
Pretreatment Unit

Figure IV-5. Sub-Potable Reuse System Flow Diagram - Class "C" Treatment

TABLE IV-8. SUB-POTABLE WATER REQUIREMENTS, TOTAL STORM WATER RUNOFF AND AMOUNT SUPPLIED

(Amounts in Gallons/Year)

			Sub-Potable Water Requirement		Total Storm Water Runoff		
Treatment Plant Location	Sub-Watersheds	Class	by Sub-watersheds Quantities	Per Treatment Plant	by Watershed	per Treatment Plant	Amount Supplied
A	1	B, C	3,369,000		3,627,000		
	2		3,401,000		4,694,000		
	3		1,420,000	8,190,000	4,073,000	12,394,000	8,190,000
B	11	B, C	8,658,000		12,378,000		
	12		4,278,000		7,332,000		
	13		6,924,000	19,860,000	10,134,000	29,844,000	19,695,000
C	4	B, C	5,212,000		4,791,000		
	5		4,923,000		7,733,000		
	14		13,724,000	23,859,000	15,528,000	28,052,000	23,856,000
E	6	C	23,327,000		17,653,000		
	8		5,497,000	28,824,000	10,741,000	28,394,000	25,555,000
D	7	C	12,238,000	12,238,000	14,007,000	14,007,000	12,238,000
F	9	B, C	17,166,000		17,818,000		
	15		2,230,000		3,700,000		
	16		10,957,000		10,260,000		
	17		12,038,000		20,388,000		
	18		3,697,000		3,905,000		
	20		12,363,000	58,451,000	20,351,000	76,422,000	58,451,000
G	19	C	1,540,000	1,540,000	15,637,000	15,637,000	1,540,000
H	22	C	1,964,000	1,964,000	5,440,000	5,440,000	1,964,000
I	21	C	7,315,000	7,315,000	11,653,000	11,653,000	7,315,000
J	10	C	12,687,000	12,687,000	25,124,000	10,049,000*	9,045,000
Totals			174,928,000	174,928,000	246,967,000	231,892,000	167,849,000

*Based on intercepting 40 percent of sub-watershed

The amount of water supplied for reuse in the Wilde Lake watershed would be 167,849,000 gallons per year. This represents 46.5 percent of the total water requirements of the area.

The pretreatment units would be operated in the same manner as previously described for the "Potable Reuse System." The probability of spills and quantities of water overflowing the reservoirs or going directly to Wilde Lake without the full Class "C" treatment would be essentially the same for the "Sub-Potable Reuse System."

With respect to maintaining adequate flow to Wilde Lake, the "Sub-Potable Reuse System" would normally discharge about 70,000,000 gallons of treated water to the Lake. This would be in addition to the base flow, possible overflows, and the 15,000,000 gallons from sub-watershed 10 that cannot be intercepted in a practical manner. Accordingly, more than adequate flow will be maintained to the Lake.

The 167,849,000 gallons of water supplied to the area represents an annual benefit of $75,532 at the current distribution price of 45 cents per thousand gallons.

4. Design and Construction Costs

The major construction items for the "Sub-Potable Reuse System" will be the same as those previously listed for the "Potable Reuse System" in Section IV.B.4 with the following exceptions:

(a) As discussed above, the treatment plants will have fewer components;

(b) The sub-potable distribution system, composed of

  (1) Underground piping

  (2) Dual Systems within buildings

Table IV-9 contains a summary of the estimated design and construction costs for the "Sub-Potable Reuse System." As shown, the cost of the sub-potable distribution system represents a major portion of the cost of the overall system. The design of the sub-potable system is based on a system of comparable design to public water systems and assumes that it is installed as a separate entity. As previously noted, in the case of new developments, this system would probably be installed concurrently with the public water and sewer system. In other cases where there is a large irrigation or industrial demand for sub-potable water a simpler or less expensive distribution system could be used. Table IV-10 illustrates the limits and effect of the cost of the distribution system.

5. Operation and Maintenance Costs

Table IV-11 contains a summary of the operation and maintenance costs for the "Sub-Potable Reuse System."

The operation and maintenance costs for the "Sub-Potable System" were estimated on the same basis as the "Potable Reuse System" with the following exceptions:

TABLE IV-9. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR SUB-POTABLE REUSE DESIGN AND CONSTRUCTION COSTS
(Amounts in Dollars)

Construction	Location										
	A	B	C	D	E	F	G	H	I	J	
Storage Reservoirs	61,000	71,500	67,500	45,000	65,000	109,000	47,000	32,500	42,000	33,500	
Pre-Treatment	8,200	9,600	10,200	9,600	9,200	11,400	8,600	7,300	8,200	7,600	
Treatment Plants	15,000	15,900	16,200			18,700					
Pumping System and Connection	12,700	11,200	13,300	5,700	5,800	27,700	5,500	5,400	5,900	7,800	
Subtotal	96,900	106,200	107,200	60,300	80,000	166,800	61,100	45,200	56,100	48,900	828,700
Distribution	37,400	134,800	143,400	230,000	100,400	380,000	43,600	25,200	57,900	97,100	
Subtotal	134,300	241,000	250,600	290,300	180,400	546,800	104,700	71,400	114,000	146,000	2,074,200
Design, Inspection and Field Engineering	21,100	36,200	37,700	43,500	27,600	81,100	15,800	10,800	17,100	21,900	
Contingencies and Escalation	13,400	24,100	25,100	29,000	18,400	54,700	10,500	7,200	11,400	14,600	
Total	168,800	301,300	312,800	362,800	225,400	682,600	136,000	87,900	142,500	182,500	2,598,600

(a) Treatment Plants

   (1) Routine inspections would require only one half man day

   (2) Routine operations of Class "C" plants would require only two man-hours per plant

(b) Distribution System

   (1) Preventive maintenance and repair at 0.6 percent per year divided equally between labor and material

TABLE IV-10. EFFECT OF DISTRIBUTION SYSTEM ON CAPITAL COSTS

System without Distribution Costs	
Construction Costs without distribution	$ 828,700.
Design, Contingencies and Escalation (25%)	207,200.
Subtotal without Distribution	1,035,900.
Equivalent Cost per Year	54,220.
Equivalent Cost per Day	148.54
System with Distribution Costs	
Construction Cost with Distribution	2,074,200.
Design Contingencies and Escalation	524,400.
Total with Distribution	2,598,600.
Equivalent Cost per Year	136,013.
Equivalent Cost per Day	372.64
Value of Sub-Potable Water per Year	75,532.
Value of Sub-Potable Water per Day	206.94

TABLE IV-11. LOCAL COLLECTION, STORAGE AND TREATMENT OF STORM WATER FOR SUB-POTABLE REUSE - ANNUAL OPERATION AND MAINTENANCE COSTS

	Labor (man hour)	Labor ($)	Materials ($)	Equipment Rental ($)	Utilities ($)	Totals
Storage Reservoirs						4,090
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690	250			
(3) Sediment Removal				2,400		
Pretreatment Units						10,050
(1) Routine Checks	260	750				
(2) Sediment Removal				3,250		
(3) Chemicals			4,500			
(4) Preventative Maintenance and Repair	520	1,730	850			
Treatment Plants						18,520
(1) Routine Inspection	1,460	4,200				
(2) Water Testing	1,040	3,060				
(3) Routine Operations	1,454	4,200				
(4) Chemicals			3,000			
(5) Preventative Maintenance and Repair	406	1,320	660			
(6) Electrical Energy					1,680	
(7) Sewer Charge					500	
Pumping Facilities						5,340
(1) Routine Checks	312	900				
(2) Preventative Maintenance and Repair	394	1,280	640			
(3) Electrical Energy					2,400	
(4) Connections	36	120				
Distribution System						7,500
(1) Maintenance and Repair	1,054	3,750	3,750			
Miscellaneous			8,670			8,670
Total	69,361	22,750	22,320	5,650	4,580	55,300

D. Local Collection, Storage, and Treatment of Storm Water for Pollution Control

1. System Description

The "Local Collection, Storage, and Treatment of Storm Water for Pollution Control" system, hereafter referred to as the "Local Pollution Control System" consists of a series/parallel arrangement of storage reservoirs which collect flows from the associated watershed and treated water and overflows from upstream reservoirs. Each storage reservoir is connected to a pretreatment unit and the water collected by the reservoir is treated to Class "C" quality and discharged into the stream beds and ultimately to Wilde Lake. Figure IV-6 shows the locations of the reservoirs and the pretreatment units. Table IV-12 lists the annual runoff from each sub-watershed, and the amount annually collected by each reservoir. Table IV-12 has been arranged to illustrate the interaction of the individual installations and the amount of flow coming from upstream reservoirs, the total flow treated annually and average per day, and the sizing of the pretreatment units and the reservoirs.

Unlike the "Potable Reuse System" and the "Sub-Potable Reuse System" described previously, the "Local Pollution Control System" makes no provision for the treatment of water beyond Class "C" quality or for distributing the water. The conceptual design for the "Local Pollution Control System" is intended to provide a basis for comparison with other pollution control systems and a reference for evaluating the benefits of reuse. The "Local Pollution Control System" has been optimized and conceptually designed as a system that could be applied for pollution control independent of reuse consideration.

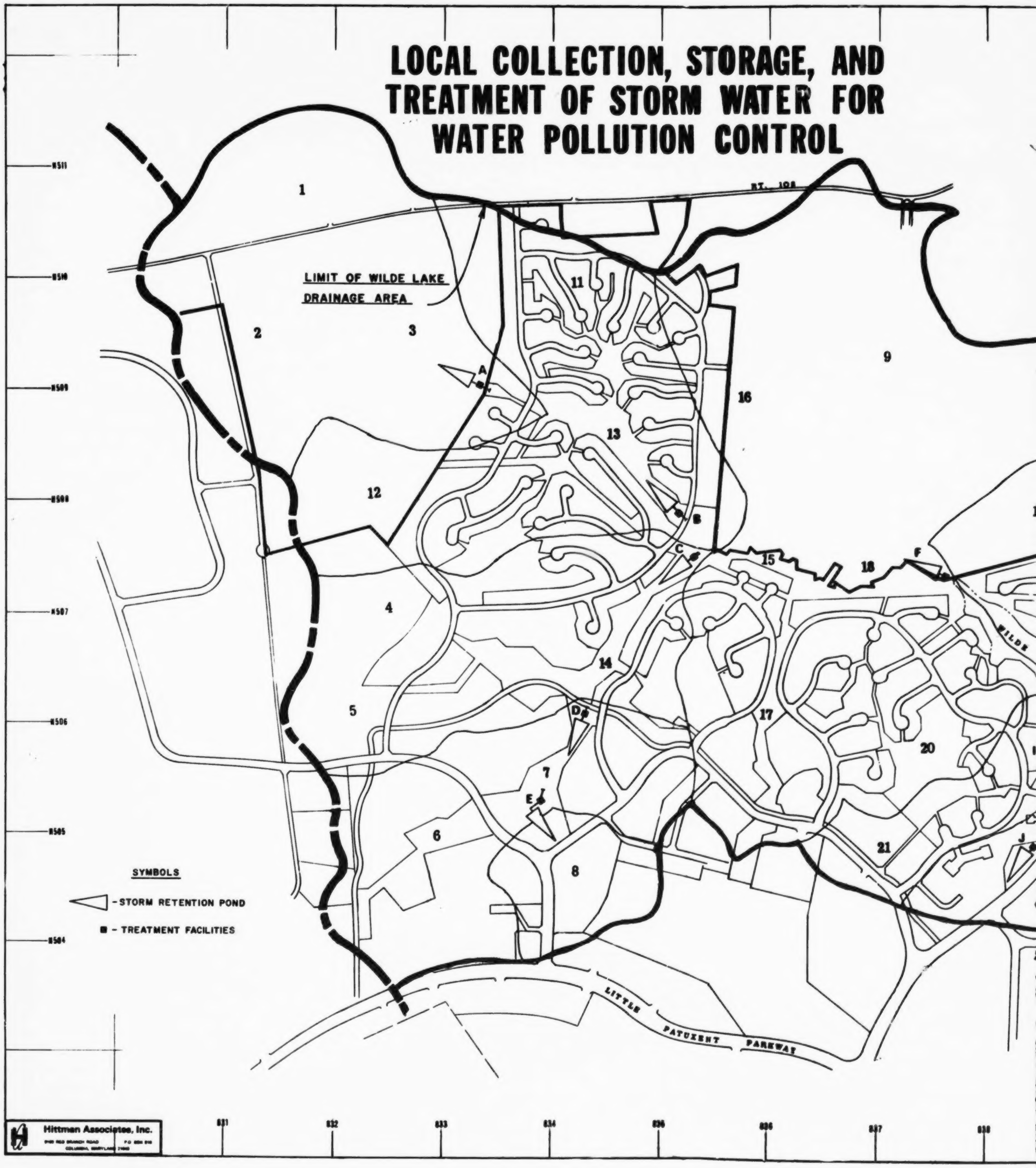
2. Component Descriptions

The "Local Pollution Control System" installations consist of two components; a storage reservoir and a pretreatment unit. These components are essentially identical in design to the storage reservoirs and pretreatment units previously described for the "Potable Reuse System" and the "Sub-Potable Reuse System." The criteria used in sizing the reservoirs and pretreatment units for the "Local Pollution Control System" are slightly different since none of the water is diverted for reuse. With the "Local Pollution Control System" flow leaving the pretreatment unit after receiving treatment to Class "C" quality is discharged directly to the stream beds.

3. Performance Characteristics

Since the sole purpose of the system is pollution control, the pretreatment units would be operated at a higher rate than the two reuse systems previously discussed. In the case of reuse systems, pretreatment flow rates would be regulated to maximize the water available for reuse. The "Local Pollution Control System" would be operated at a rate of twice the mean annual daily flow rate to increase the probability of that the reservoirs will be able to collect the majority, if not all, of subsequent storms. As discussed in Section IV-B-3, with the pretreatment

LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR WATER POLLUTION CONTROL



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SYMBOLS
- STORM RETENTION POND
■ - TREATMENT FACILITIES

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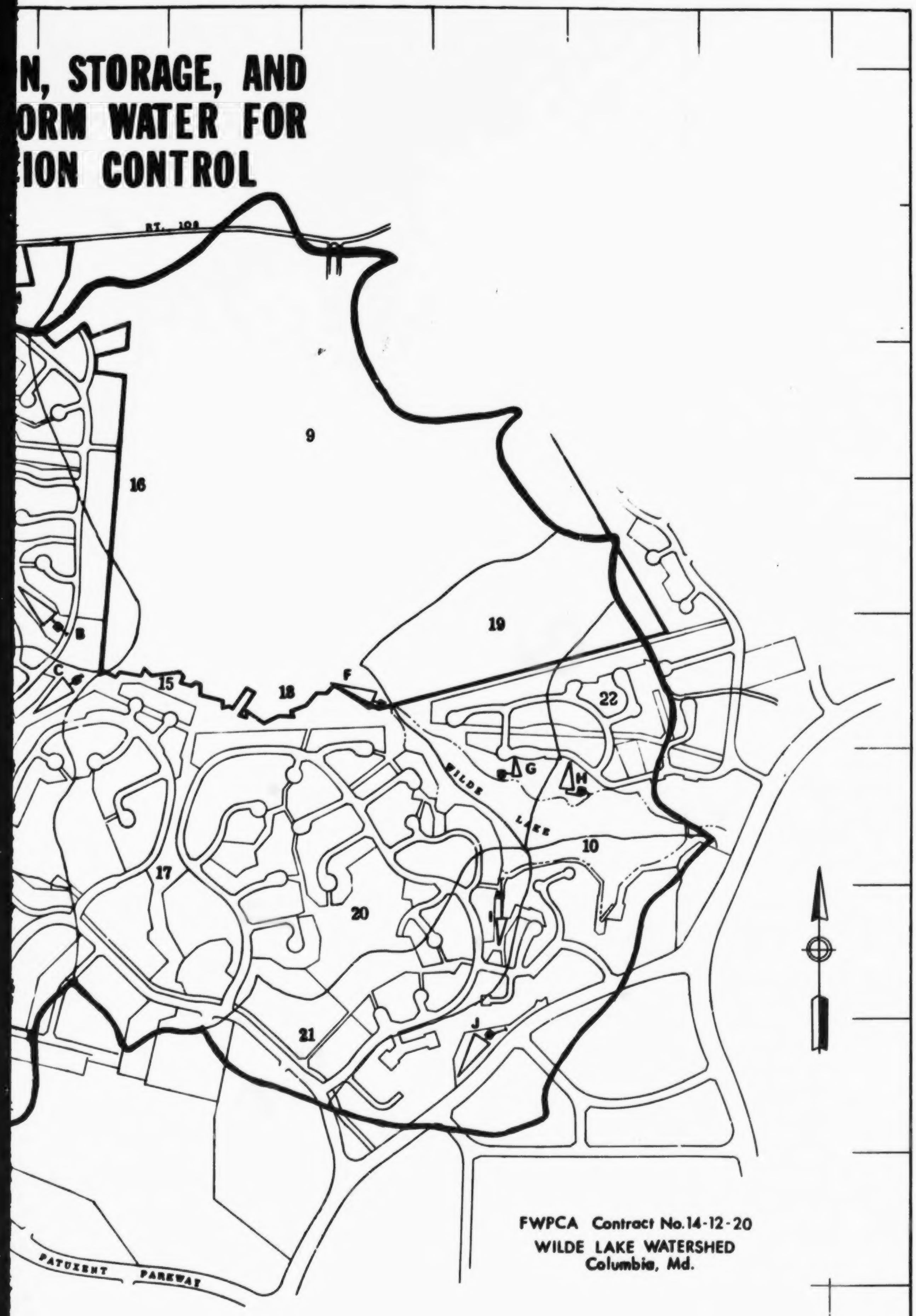
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WILDE LAKE WATERSHED
Columbia, Md.
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Figure IV-6. Local Pollution Control System - Location Plan

TABLE IV-12. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR POLLUTION CONTROL RUNOFF BY SUB-WATERSHED AND PRETREATMENT UNIT WITH PRETREATMENT AND RESERVOIR CAPACITIES

Reservoir/ Pretreatment Location	Sub-Watershed Runoff		Total Runoff Per Reservoir (gpy)	Flow from		Total Flow Treated		Pretreatment Capacity (gpd)	Reservoir Size (gals)
	No.	(gpy)		Upstream No.	Reservoir (gpy)	(gpy)	(gpd)		
A	1	3,627,000							
	2	4,694,000							
	3	4,073,000	12,394,000			12,394,000	34,000	68,000	1,577,300
B	11	12,378,000							
	12	7,332,000							
	13	10,134,000	29,844,000	A	12,394,000	42,238,000	116,000	232,000	2,145,000
E	8	10,741,000	10,741,000			10,741,000	28,500	59,000	1,729,000
D	6	17,653,000							
	7	14,007,000	31,660,000	E	10,741,000	42,401,000	116,200	232,400	803,000
C	4	4,791,000							
	5	7,733,000							
	14	15,528,000	28,052,000	D	42,401,000	70,454,000	193,000	386,000	1,902,000
F	9	17,818,000							
	15	3,700,000							
	16	10,260,000							
	17	20,388,000							
	18	3,905,000		B	42,238,000				
	20	20,351,000	76,422,000	C	70,454,000	189,113,000	518,000	1,036,000	5,101,000
G	19	15,637,000	15,637,000			15,637,000	43,000	86,000	923,500
H	22	5,440,000	5,440,000			5,440,000	14,900	29,800	358,400
I	21	11,653,000	11,653,000			11,653,000	32,000	64,000	690,000
J	10	25,124,000	10,049,000*			10,049,000*	26,000	53,200	375,000*
		246,967,000							

*Based on intercepting 40 percent of the watershed unit operating at twice the mean annual runoff plus base flow, 98.5 percent of all "one-year" storms will be stored. At this rate 91.2 percent of all storms, including those with return intervals greater than one year, will be retained.

With the "Local Pollution Control System" only that water from sub-watershed number 10 will enter Wilde Lake untreated. This amounts to 15,075,000 per year and of the remaining 231,892,000 gallons of annual runoff, only 3,478,000 gallons will not receive the full Class "C" treatment for any normal year and only 20,407,000 gallons for storms of greater return intervals. All water except that from sub-watershed 10 will be retained in the storage reservoirs for a sufficient interval to permit the heavy particles to settle out. Expressed differently, if the water not receiving full treatment were to have a suspended solids concentration of 200 ppm, the resultant overall suspended solids in the lake would be 43 ppm when diluted with the treated effluent having 30 ppm suspended solids, assuming no sedimentation in Wilde Lake itself.

4. Design and Construction Costs

The construction items for the "Local Pollution Control System" would be the same as the two reuse systems previously discussed with respect to the storage reservoirs and pretreatment units. Table IV-13 contains a summary of the estimated design and construction costs for the "Local Pollution Control System."

5. Operation and Maintenance Costs

The operation and maintenance for the "Local Pollution Control System" would involve the same items of work as the storage reservoirs and pretreatment units for the two reuse systems previously discussed. Table IV-14 contains a summary of the estimated costs for the "Local Pollution Control System."

TABLE IV-13. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR WATER POLLUTION CONTROL - DESIGN AND CONSTRUCTION COSTS

	Location										
	A	B	C	D	E	F	G	H	I	J	Total
Construction											
Storage Reservoirs	61,000	71,500	67,500	45,000	65,000	109,000	47,000	32,500	42,000	33,500	
Pretreatment Units	8,200	9,600	10,200	9,600	9,200	11,400	8,600	7,300	8,200	7,600	
Subtotal	69,200	81,100	77,700	54,600	74,200	120,400	55,600	39,800	50,200	41,100	663,900
Design, Inspection and Field Engineering	10,400	12,100	11,700	8,200	11,100	18,000	8,400	6,000	7,500	6,100	99,500
Contingencies and Escalation	6,900	8,100	7,800	5,500	7,400	12,000	5,600	4,000	5,000	4,100	66,400
Total	86,500	101,300	97,200	68,300	92,700	150,400	69,600	49,800	62,700	57,300	829,800

TABLE IV-14. LOCAL COLLECTION, STORAGE, AND TREATMENT OF STORM WATER FOR WATER POLLUTION CONTROL - ANNUAL OPERATION AND MAINTENANCE COSTS

	Labor (man hrs.)	Labor ($)	Material ($)	Equipment Rental ($)	Utilities ($)	Totals
Storage Reservoirs						
(1) Routine Checks	260	750				
(2) Trash Removal and Nuisance Control	240	690				
(3) Sediment Removal				2,400		
Pretreatment Units						
(1) Routine Checks	260	750				
(2) Sediment Removal				3,250		
(3) Chemicals			4,500			
(4) Preventative Maintenance and Repair	520	1,730	1,730			
(5) Electrical Energy					200	
Miscellaneous			3,600			
Subtotals	1,280	4,320	9,830	5,650	200	$20,000

V. CONVENTIONAL STORM WATER TREATMENT SYSTEM

A. Introduction

In order to evaluate the systems for "Local Storage, Treatment, and Reuse of Storm Water" it was necessary to develop a base for comparison. As a parallel effort, a conceptual design was prepared for a system to control the pollution of Wilde Lake using conventional design methods. This work was performed by Whitman, Requardt and Associates of Baltimore, Maryland, and a copy of their report is contained in Appendix D. A summary description of the Conventional Storm Water Treatment System, hereafter referred to as the "Conventional Treatment System," is contained in this chapter. In addition to developing the design of a conventional system as a basis for comparison, surveys were made of the experience with other artificial lakes, similar to Wilde Lake. Section V.F of this chapter contains information on the efforts and costs involved in maintaining the quality of Lake Barcroft in Fairfax County, Virginia.

B. Design Requirements

The design of the Conventional Treatment System was based on the same area, runoff quantities, flow hydrographs, and same quality of water requirements as the local storage, treatment, and reuse system. Table V-1 contains a summary of the design requirements:

TABLE V-1. DESIGN REQUIREMENTS FOR CONVENTIONAL TREATMENT PLANT

Area Considered:	Wilde Lake Watershed
Treated Water Quality:	State of Maryland Water Quality Standards for "Water Contact Recreation" Modified as discussed in Section II.D under Class "C" Water Quality
Water Quantity:	Runoff quantities as calculated using the synthesized hydrograph method with storms of one year return interval as the basis for design

C. System Description

The Conventional Treatment System consists, in part, of a central treatment basin located at the head of Wilde Lake and the necessary facilities to collect and pump the runoff water to the treatment basin. As shown on Figure V-1, the Conventional Treatment System is divided by natural boundaries into two parts. "Area A" consists of 954 acres of the watershed from which water flows by gravity to the treatment basin. "Area B" consists of an area of 165 acres from which runoff must be collected and pumped to the treatment basin. The components comprising the Conventional Treatment System are as follows:

(1) Holding Pond. This pond is located at the east side of Area B and has a capacity of 1,840,000 gallons. The pond receives the runoff from the major portions of sub-watersheds 10, 20, and 21 from a gravity interceptor on the west side of the pond. The holding pond provides storage capacity in order to avoid pumping runoff water at storm rates.

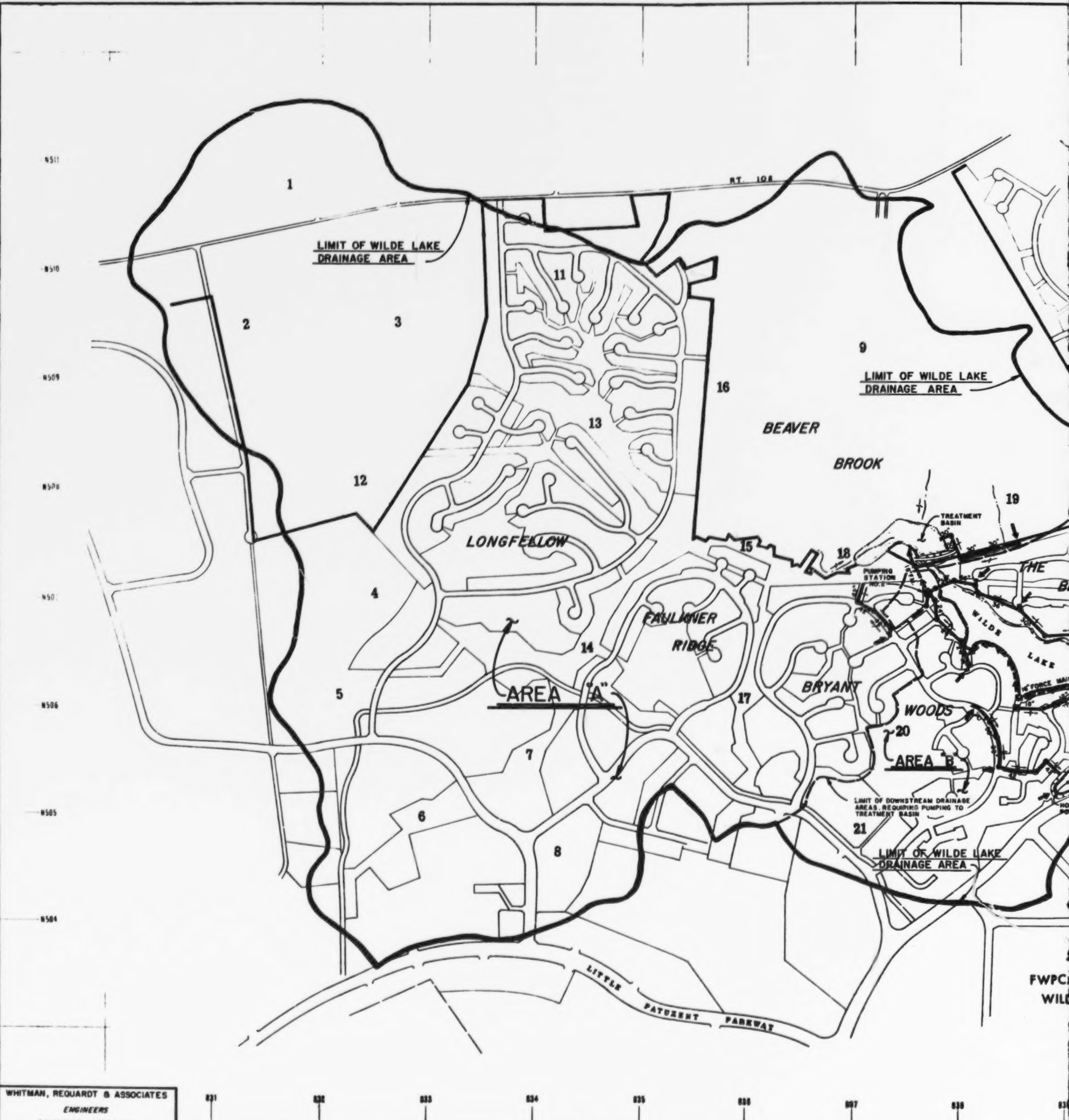
(2) Pumping Station No. 1. This station is located on the south side of Wilde Lake and pumps the water from the holding pond to Pumping Station No. 2 for transfer to the treatment basin. This station also receives flow from gravity interceptors which collect runoff from the watershed adjacent to the south side of Wilde Lake. This station has a pumping capacity of 2.5 mgd.

(3) Gravity Interceptors. The northern most gravity interceptor collects the runoff from the majority of sub-watersheds 9, 16, and 19, and is located to permit this water to flow by gravity directly to the treatment basin. The gravity interceptor located adjacent to the north side of Wilde Lake collects the runoff from the lower portions of the sub-watersheds, which flows by gravity to the wet well of Pumping Station No. 2.

A gravity interceptor located at the west of the head of Wilde Lake intercepts the majority of the flow from sub-watershed 17 in order for the water to flow by gravity to the treatment basin. A second gravity interceptor located adjacent to the southwest side of Wilde Lake collects the runoff from the lower portion of the watershed and which flows by gravity to Pumping Station No. 2.

(4) Pumping Station No. 2. This station receives runoff from the two gravity interceptors located adjacent to the north and southwest side of Wilde Lake. This station also transfers the water from the force main from Pumping Station No. 1 to the treatment basin. Pumping Station No. 2 uses dual 5 mgd pumps for a maximum capacity of 10 mgd.

(5) Treatment Basis. The treatment basin is located at the head of Wilde Lake as shown on Figure V-1. Figure V-2 is a more detailed drawing of the treatment basin. The treatment basin


N511
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RT. 108
1
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17
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19
20
21
LIMIT OF WILDE LAKE DRAINAGE AREA
LIMIT OF WILDE LAKE DRAINAGE AREA
LIMIT OF WILDE LAKE DRAINAGE AREA
BEAVER BROOK
LONGFELLOW
FAULKNER RIDGE
BRYANT WOODS
AREA "A"
AREA "B"
TREATMENT BASIN
PUMPING STATION NO. 2
WILDE LAKE
THE
FORCE MAIN
LIMIT OF DOWNSTREAM DRAINAGE AREAS, REQUIRING PUMPING TO TREATMENT BASIN
LITTLE PATUXENT PARKWAY
FWPC
WIL
831
832
833
834
835
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838

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BALTIMORE, MARYLAND

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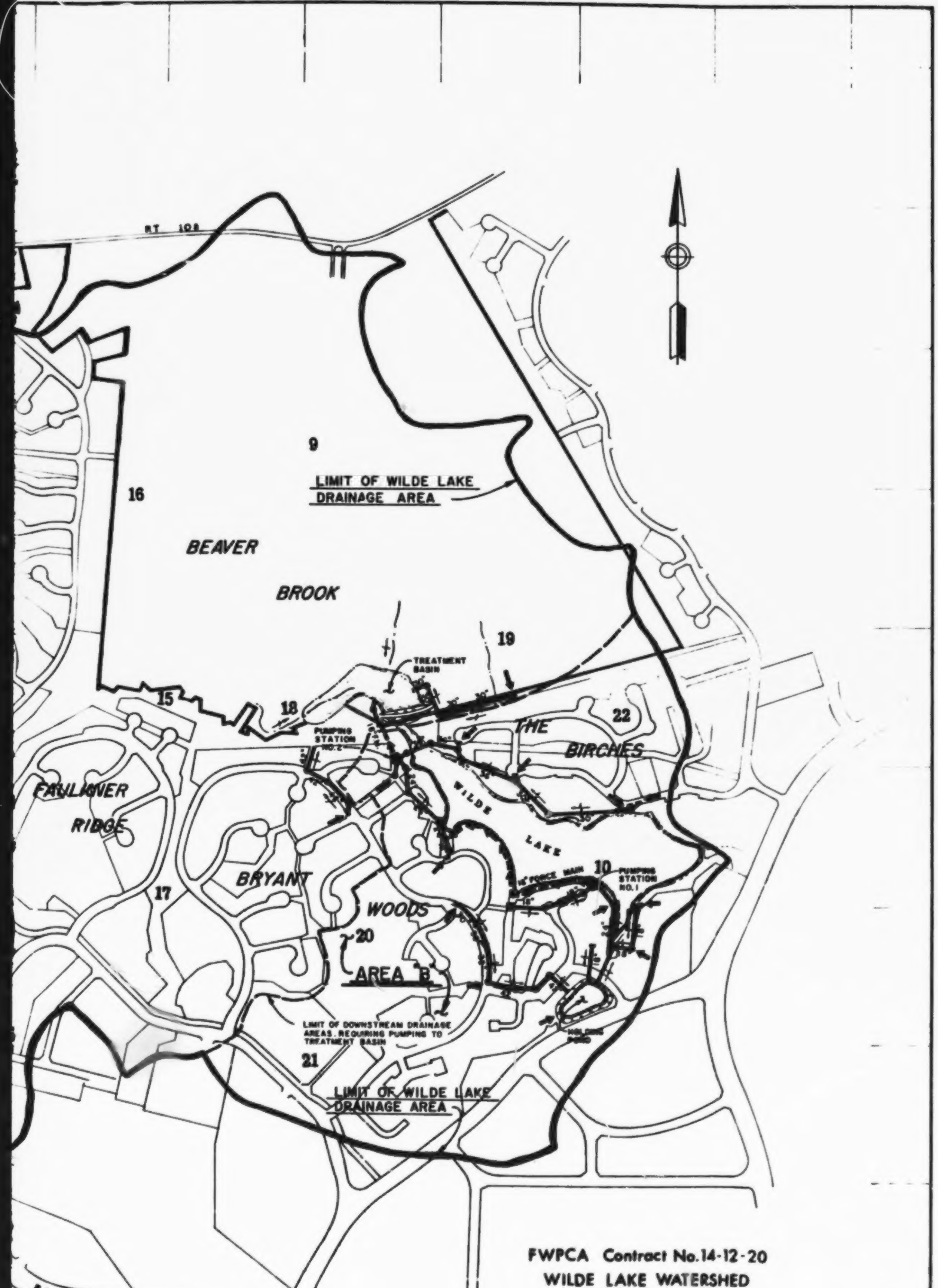
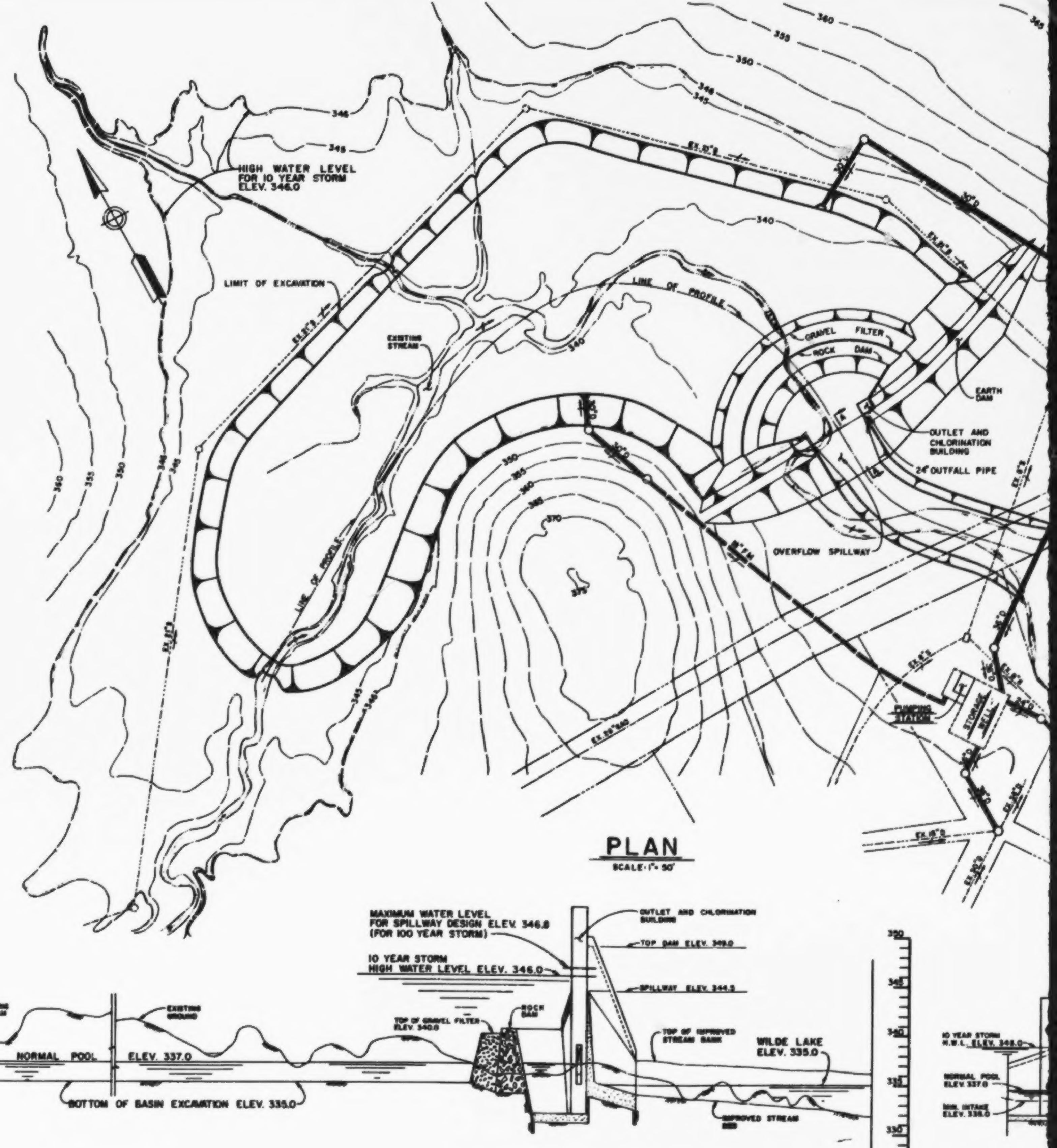

RT. 108
9
LIMIT OF WILDE LAKE DRAINAGE AREA
16
BEAVER BROOK
19
TREATMENT BASIN
15
18
PUMPING STATION NO. 2
THE BIRCHES
22
FAULKNER RIDGE
WILDE LAKE
BRYANT WOODS
17
10
FORCE MAIN
PUMPING STATION NO. 1
20
AREA B
LIMIT OF DOWNSTREAM DRAINAGE AREAS, REQUIRING PUMPING TO TREATMENT BASIN
HOLDING POND
21
LIMIT OF WILDE LAKE DRAINAGE AREA
PATUXENT PARKWAY
FWPCA Contract No. 14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
835
836
837
838
839
PLAN OF CONVENTIONAL SYSTEM
SCALE: 1" = 400'

Figure V-1. Plan of Conventional Treatment System

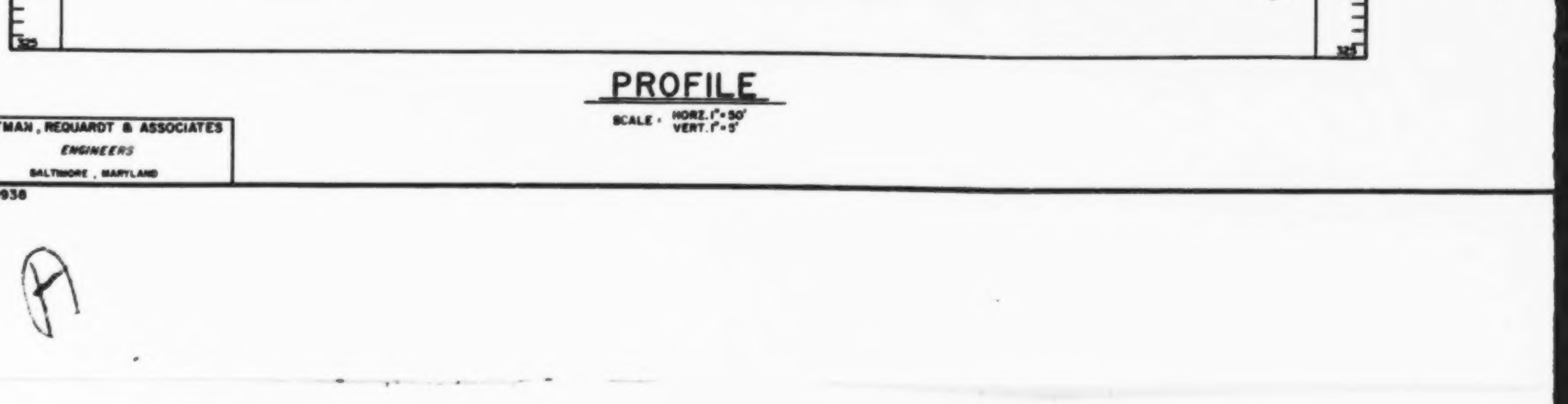
B



360
355
350
348
345
340
365
HIGH WATER LEVEL FOR 10 YEAR STORM ELEV. 346.0
LIMIT OF EXCAVATION
EXISTING STREAM
LINE OF PROFILE
GRAVEL FILTER
ROCK DAM
EARTH DAM
OUTLET AND CHLORINATION BUILDING
24" OUTFALL PIPE
OVERFLOW SPILLWAY
PUMPING STATION
STORAGE WELL
370
375

PLAN

SCALE: 1" = 50'



MAXIMUM WATER LEVEL FOR SPILLWAY DESIGN ELEV. 346.8 (FOR 100 YEAR STORM)
OUTLET AND CHLORINATION BUILDING
TOP DAM ELEV. 349.0
10 YEAR STORM HIGH WATER LEVEL ELEV. 346.0
SPILLWAY ELEV. 344.5
EXISTING STREAM BED
EXISTING GROUND
TOP OF GRAVEL FILTER ELEV. 340.0
ROCK DAM
TOP OF IMPROVED STREAM BANK
WILDE LAKE ELEV. 335.0
NORMAL POOL ELEV. 337.0
BOTTOM OF BASIN EXCAVATION ELEV. 335.0
IMPROVED STREAM BED
10 YEAR STORM H.W.L. ELEV. 346.0
NORMAL POOL ELEV. 337.0
MIN. INTAKE ELEV. 336.0
350
345
340
335
330
325

PROFILE

SCALE: HORZ. 1" = 50'
VERT. 1" = 5'

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BALTIMORE, MARYLAND

W.O. 5938

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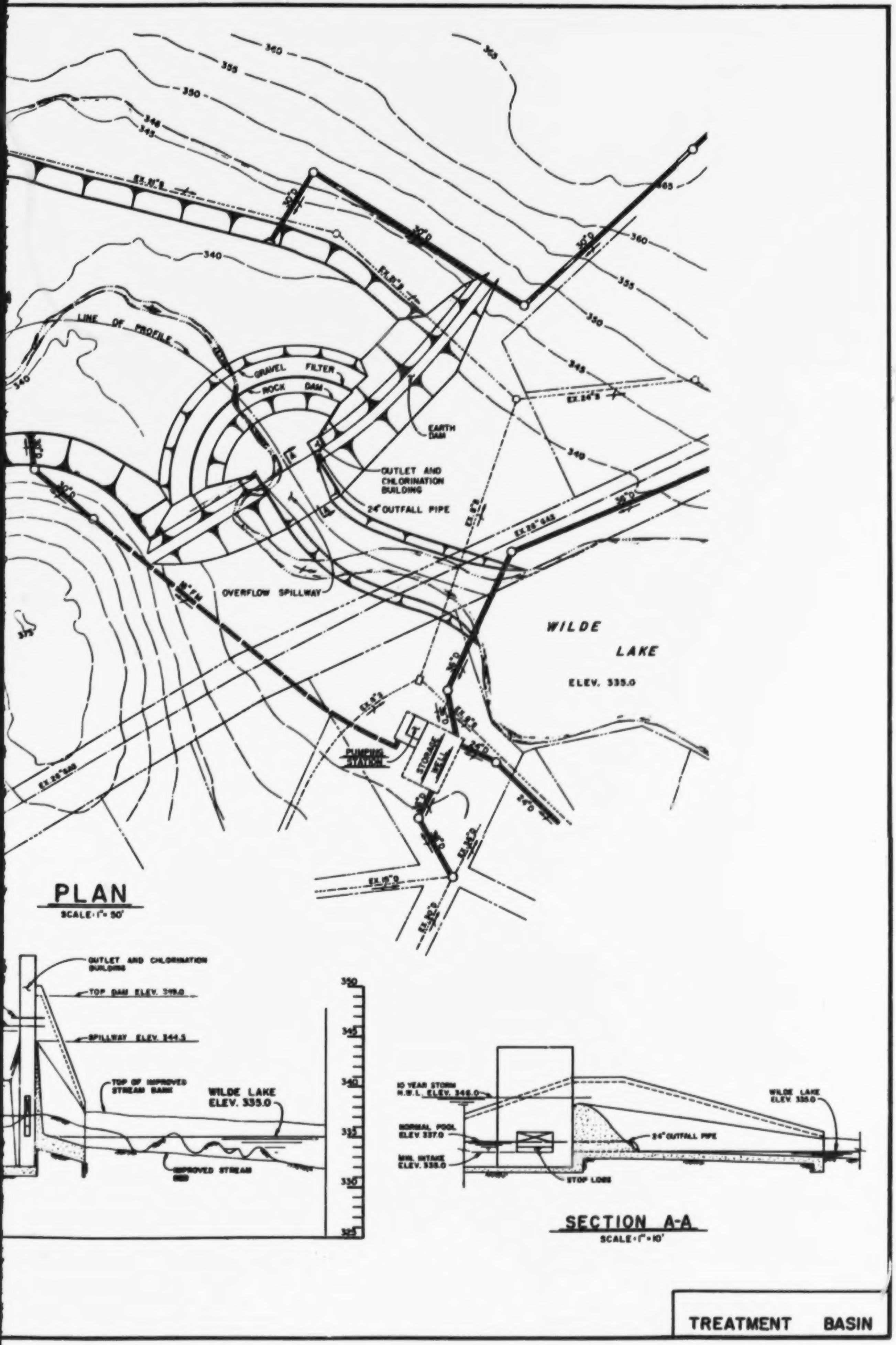
GRAVEL FILTER
ROCK DAM
LINE OF PROFILE
EARTH DAM
OUTLET AND CHLORINATION BUILDING
24" OUTFALL PIPE
OVERFLOW SPILLWAY
WILDE LAKE
ELEV. 335.0
PUMPING STATION
STORAGE WELL
PLAN
SCALE: 1" = 50'
OUTLET AND CHLORINATION BUILDING
TOP DAM ELEV. 349.0
SPILLWAY ELEV. 344.5
TOP OF IMPROVED STREAM BANK
WILDE LAKE ELEV. 335.0
IMPROVED STREAM BED
10 YEAR STORM H.W.L. ELEV. 346.0
NORMAL POOL ELEV. 337.0
MIN. INTAKE ELEV. 335.0
STOP LOGS
24" OUTFALL PIPE
WILDE LAKE ELEV. 335.0
SECTION A-A
SCALE: 1" = 10'
TREATMENT BASIN

B Figure V-2. Conventional System Treatment Basin consists of an earth and concrete dam and an excavated basin sized to retain a one year storm. The treatment basin provides retention of the storm water runoff to allow sedimentation of suspended solids and incorporates a rock dam and gravel filter for filtration of water. Chlorination equipment is provided to disinfect the effluent and is designed to provide variable dosages to meet the range of overflow rates. The normal pool elevation is maintained by stop logs in the outlet structure and the pool may be drained for silt removal and other purposes by removal of the logs. The principal features of the treatment basin are as follows:

Dam height	- 15 feet from existing stream bed to top of dam elevation 349.0
Spillway length	- 60 feet
Spillway overflow elevation	- 344.5
One year storm maximum water level	- 344.0
Normal pool storage	- 2.6 MG
Maximum storm storage	- 12.4 MG from normal pool to spillway overflow
High water level	- 346.0 (10 year storm)
High water volume	- 16.6 MG from normal pool to high water level
Area at high water level	- 424,000 square feet (9.75 acres)
Maximum water level for dam design	- 346.8 (100 year storm)

The outlet from the treatment basin is a 24 inch conduit which will drain the basin in 19 hours at an overage flow of 20 MGD varying from 27.5 MGD at the high water level elevation to 0.5 MGD at the low pool elevation.

A more detailed discussion of the basis for the design of the Conventional Treatment Systems is contained in Appendix D.

D. Performance Characteristics

The Conventional Treatment System is designed to collect and transfer storm water runoff to the treatment basin at the runoff rates and quantities of a one year storm. The system is designed to drain the holding pond and the treatment basin to the minimum levels within a 24 hour period in order to minimize the probability that these storage basins will not be sufficiently empty to receive subsequent storms. The treatment basin is designed to meet the State of Maryland requirements for a Class 3 impoundment and the dam and spillway design is based on a 100 year storm. However, at storms of greater than one year return interval, the capacity of gravity interceptors and the pumping stations may be exceeded and a portion of the runoff will overflow directly to Wilde Lake. All of the runoff flowing directly to the treatment basin would be retained for some period.

The treatment provided the storm water with the Conventional Treatment System consists of sedimentation, gravel filtration, and chlorination. The removal of suspended solids and resulting reduction of turbidity in storm water by natural sedimentation is a function of the water surface area and overflow rate of the reservoir. Table V-2 contains a listing of the size particles that would be removed with the treatment basin used for the Conventional Treatment System.

TABLE V-2. PARTICLE REMOVAL WITH CONVENTIONAL TREATMENT BASIN

Water Elevation	Effluent Q (MGD)	Water Surface Area (square ft.)	Overflow Rate (gal/day/sf)	Size of Particles Removed (Microns)
337.0	10.0	181,000	55	7.5
338.0	14.0	186,000	75	8.8
340.0	19.5	196,000	100	10.1
342.0	24.0	219,000	110	10.6
344.5	28.0	320,000	89	9.5
346.0	278.0	424,000	660	26.0

Table V-2 shows that the treatment basin will remove silt particles (approximately 10 microns) for overflow rates up to the spillway (elevation 344.5 feet) which includes the one year storm (elevation 344.0 feet). However, it is considered that "Class C" water will require a greater reduction of turbidity and settlement of particles smaller than 10 microns.

In order to provide for treatment to a quality comparable to "Class C" the treatment basin incorporates a rock dam backed by a gravel filter through which storms of less than 5,000,000 gallons of runoff would pass, as would at least this amount of runoff from storms greater than 5,000,000 gallons. Storms of 5,000,000 gallons magnitude are exceeded six percent of the time. In terms of quantity of runoff, the 5,000,000 gallon filtration capability would represent about 60 percent of the total.

A storm of 18,600,000 gallons per day on the entire drainage will utilize 12,400,000 of the storage capacity of the basin without overtopping the spillway. For the five years of record, storms of 18,600,000 gallons per day were exceeded 0.25 percent of the time, which represents two to three percent of the total flow.

E. Design and Construction Costs

As part of the conceptual design of the Conventional Treatment System, an estimate of the design and construction costs was prepared. Table V-3 contains a summary of this estimate and further details are contained in Appendix D.

TABLE V-3. DESIGN AND CONSTRUCTION COSTS CONVENTIONAL TREATMENT SYSTEM

Construction Costs		
Holding Pond	$ 24,000	
Pumping Station No. 1	117,000	
Force Main	30,000	
Interceptor System	205,000	
Pumping Station No. 2	300,000	
Treatment Basin	163,000	
Subtotal		$ 939,000
Design, Field Engineering & Inspection		187,800
Contingencies and Escalation		187,800
TOTAL		$1,314,500

The $1,314,500 construction cost for the Conventional Treatment System translates to a daily cost of $198.50 for the system using the same amortization rate previously used with the local storage, treatment, and reuse system

F. Operation and Maintenance Costs

The operating and maintenance costs for the Conventional Treatment System were estimated and details of this estimate are contained in Appendix D. The operating and maintenance cost is estimated at $25,000 per year which translates to $68.49 per day. The summary in Appendix D includes an allowance for depreciation of mechanical equipment which was deleted here to maintain consistent cost estimating practices.

G. Lake Barcroft, Fairfax County, Virginia

The Conventional Treatment System discussed earlier in this chapter represents an approach that could be used to control the pollution of Wilde Lake using conventional methods and technology. Another approach that can be used to provide a comparative base for the local storage, treatment, and reuse system would be to analyze the methods that have been used in other areas to control pollution and to maintain the water quality and the value of lakes and adjacent property.

A limited survey was made to determine whether any existing situations existed which could be used for comparison. The one case found of most direct interest is Lake Barcroft in Fairfax County, Virginia. (References 44 and 45).

Lake Barcroft was originally constructed as a water supply reservoir for the city of Alexandria, Virginia. The reservoir receives drainage from the Tripps Run watershed which is a 5.5 square mile area and the Holmes Run watershed which is a 7.6 square mile area. In addition, there is a 1.5 square mile area which drains directly to the lake. The lake comprises an area of 118 acres with water depths varying from 5 to 40 feet.

A comprehensive study of the siltation problems of Lake Barcroft was made in 1957 by the Soil Conservation Service of the U. S. Department of Agriculture. This study estimated that the capacity loss of Lake Barcroft for the period 1957-1970 would be approximately 6.5 acre feet per year. This study indicated that the period of maximum silt deposition was the ten year period 1945-1955 when urban development in the watershed proceeded at a highly accelerated rate. It was estimated that during this period 225,000 cubic yards of silt was deposited in the lake of which an estimated 80 percent was brought in by high intensity flood flows.

Lake Barcroft is no longer used as a water supply reservoir and the loss of capacity is not important as it would be in a supply reservoir. The lake is presently used for recreational activities in a high-value residential neighborhood and the siltation problem affects not only the lake shore property owners but the community in general.

A study was made of the flood control and storm drainage of the area in 1958 and improvements were made in the lake in 1959 to 1961. These improvements consisted of increasing the depth of the lake by dredging and the construction of siltation basins in the channels of Holmes Run and Tripp Run. The depth of the Tripps Run basin is approximately eight feet below the low water level and it has a capacity of 22,000 cubic yards. Rock was encountered on the Holmes Run basin and it is only three to four feet below the low water level and has a capacity of about 12,000 cubic yards.

The first silt removal project from the basins was conducted in 1963. The water level was lowered and approximately 6,000 yards of silt was removed from each basin with draglines and front end loader. An additional 6,000 yards of material was removed from other areas and the total cost of the project was about $40,000.

Another silt removal project was begun in 1965. In this project, about 6,000 cubic yards of material were removed from the Holmes Run basin and about 5,500 cubic yards from the Tripps Run basin. In the 1963 silt removal project, underwater roads had been constructed to facilitate the silt removal operation. These were extended during the 1965 operation. The total cost of removal of material from the basins was approximately $27,000, not including material removed from other areas and the lower cost was attributed to the underwater roads. A third silt removal project is being planned for late 1968. It is estimated that this operation will require about six weeks.

The experience with silt removal at Lake Barcroft has shown several problems and difficulties. The silt is in large portion an extremely wet, partly organic, near-colloidal material. The disposal areas must have adequate drainage to allow the silt to dry and locating of suitable areas has been a problem. The wet silt is similar to quicksand and presents a safety hazard to children during the drying period. Frequent road maintenance has been necessary due to spillage from the trucks transporting wet silt through the area. Due to limits on the reach of dragline crane booms, it has been necessary to construct underwater roads. The maintenance costs of construction equipment used in silt removal operations is higher than normal and there is a hesitancy on the part of contractors to provide equipment for silt removal operations.

Lake Barcroft is a privately owned lake and is maintained by the Lake Barcroft Community Association. The community, through the Lake Barcroft Small Sanitary Districts, assesses property owners at a rate of $0.25 per $100 of assessed valuation to provide funds for the maintenance of the lake and other storm water drainage, roads, and other improvements in the area.

During this limited survey, the cost of constructing the siltation basins and the underwater roads was not determined. However, based on the silt removal costs of $2.20 to $2.40 per cubic yard, the basins, which have a combined capacity of 34,000 cubic yards, the probable costs is in the range of $75,000 to $100,000. The direct costs of the silt removal operation have been averaging about $15,000 per year and the total annual cost greater than $25,000 considering the construction of the basins and the roads. There are 400 to 500 residences in the Lake Barcroft community having lots of about one acre. This would indicate an annual cost of $30 to $60 per dwelling unit per year and approximately the same cost per acre per year.

A direct comparison of the cost of silt removal from Lake Barcroft with the cost of the Local Pollution Control System or the Conventional Treatment System considered in this study is not possible. First, the Lake Barcroft watershed is much larger and the cost noted above considers only the residences in the immediate area of the lake. The total cost of storm water and flood control in the entire watershed and the number of dwelling units served would have to be determined to make a direct comparison. The siltation basins at Lake Barcroft only remove the heavy sediment and do not provide the same quality of water as either the Local or Convention Pollution Control Systems considered in this study. Table V-2 and the discussion in Appendix D indicates the sediment removal capabilities of reservoirs at high flow rates. The siltation basins at Lake Barcroft receive flow from the major tributaries to the lake and do not attempt to treat all flows to the lake.

VI. SYSTEM EVALUATION

A. Introduction

Chapters II and III described the system analysis and optimization of the local storage, treatment, and reuse of storm water system. An actual watershed in Columbia, Maryland was used to develop optimum conceptual systems corresponding to three different assumptions concerning reuse:

(1) Reuse as potable water

(2) Reuse as sub-potable water

(3) No reuse with the system operated as water pollution control measure only

Descriptions and performance characteristics of the three conceptual systems are presented in Chapter IV.

Whitman, Requardt and Associates, a Baltimore consulting engineering firm, was engaged as a subcontractor and charged with the preliminary design of a facility to accomplish the same water pollution control objective set for the local reuse system. The subcontractor was free to employ any method they considered effective and appropriate, provided they limited themselves to present technology. The design proposed by Whitman, Requardt and Associates is referred to in this report as the "Conventional Storm Water System Design" and is described and evaluated in Chapter V and Appendix D. The purpose of this effort was to provide a realistic framework for evaluation of the local reuse concept. To the extent that the conventional system represents the best alternative method of accomplishing the same objective, its costs can be taken as the imputed value of water pollution control. For the purposes of this study, such an approach is considered more satisfactory than attempts to quantify economic losses associated with storm water pollution, or conversely, economic benefits expected from pollution abatement.

The following sections represent a detailed evaluation of the local reuse concept, taking into account its generalized characteristics as well as specific features of importance to the Columbia location. Economic comparisons and performance estimates have been based on the characteristics of the alternative conventional system.

B. Economic Comparison

1. Potable Reuse

Capital costs associated with the optimum potable reuse system were detailed in Chapter IV and found to total, after adjustment, $1,445,200. When these costs are amortized over the life of the facilities by the method described in Section II. A, capital costs are found to be equivalent to uniform costs equalling $207.21 per day. Operating and maintenance costs, detailed in Chapter IV, are estimated at $194.10 per day. The sum of capital, operating and maintenance costs is the total system cost and can be found to equal $401.31 per day, over the estimated life of the facilities. Offsetting these costs are tangible benefits from two sources. The treated water which is produced for reuse has been evaluated at $0.45 per thousand gallons and totals $278.00 per day in the case of potable reuse. The total system cost of the conventional storm water treatment system was computed as $256.99 per day (based on capital costs of $1,314,500 and annual operating and maintenance costs of $25,000). Taking the imputed value of water pollution control as the total system cost of the best available alternative, total benefits can be computed as the sum of the value of reused water and the cost of conventional treatment, or $534.99 per day. These computations are summarized on Table VI-1.

This computation of benefits and costs can be used to evaluate a benefit/cost ratio of 1.26 or a daily net benefit of $105.88 depending on the purpose of the analysis. Several other factors bearing on this analysis should be considered in the interest of a comprehensive evaluation. The benefit/cost summary just outlined is appropriate to a water resource project having a variety of purposes and potential benefits. Two of the principal benefits, water supply and water pollution control, have been quantified and compared to total system costs. Other intangible benefits are discussed in later sections of this chapter and must be considered as part of the evaluation. The costs and benefits presented above can be organized somewhat differently in the case of projects intended principally as water supply facilities, or as water pollution control facilities. Table VI-1 includes analyses of this type and lists the cost of potable water after full credit is taken for the water pollution control aspect of the system, as well as the cost of water pollution control after full credit is taken for the value of the potable water. Both of these costs are shown in units which permit ready comparison to other cost indices.

Critical to the calculation of benefit/cost ratios is the method used to evaluate benefits. The benefits noted above and on Table VI-1 are computed consistent with the assumptions decribed in other sections of this report. The sensitivity of the results to these assumptions can be investigated rather easily. The value of reused water, for example, has been taken at $0.45 per thousand gallons. Should this water be purchased by the local water utility and distributed and sold by them, the value of the treated water should conform to the cost to the water utility of an alternative supply. In the case of the Howard County Metropolitan Commission, this would be the incremental cost of water purchased from Baltimore City as well as other incremental costs associated with pumping, storing, and conveying that water to the Wilde Lake watershed. An exact determination of this cost would require a thorough study of the financial structure of the Metropolitan Commission.

TABLE VI-1. ECONOMIC ANALYSIS OF POTABLE REUSE SYSTEM VS. CONVENTIONAL STORMWATER TREATMENT

Costs	
Capital Cost of System	$ 1,445,200.
Amortized Capital Cost ($/Day)	207.21
Operating and Maintenance Cost ($/Year)	70,845.00
Operating and Maintenance Cost ($/Day)	194.10
Total System Cost ($/Day)	401.31
Benefits	
Water Reuse	
Quantity of Reused Water (Gallons/Year)	202,942,000.
Value of Reused Water ($/Year@$0.45/1000 Gallons)	91,324.00
Value of Reused Water ($/Day)	250.20
Water Pollution Control	
Capital Cost of Conventional System	1,314,500.
Amortized Capital Cost ($/Day)	188.50
Operating and Maintenance Cost ($/Year)	25,000.
Operating and Maintenance Cost ($/Day)	68.49
Total System Cost ($/Day)	256.99
Total Benefits ($/Day)	507.19
Multipurpose Water Resource Project	
Net Benefits ($/Day)	105.88
Benefit/Cost Ratio	1.26
Water Supply Project	
Total Cost Less Water Pollution Benefit ($/Year)	52,680.
Net Cost of Treated Water ($/1000 Gallons)	0.26
Water Pollution Control Project	
Total Cost Less Water Supply Benefit ($/Year)	55,160.
Net Cost of Pollution Control ($/Acre-Year)	48.05
($/DU*-Year)	14.68

*DU = Dwelling Unit

Another consideration might be the effect of higher water values, such as those existing in other areas of the country, on the economic evaluation. A comparison of this type is of limited value, since the system cost figures refer to a specific installation at Columbia, Maryland and costs might vary widely in different locations. Figure VI-1 shows the relationship of the Net Benefit measure developed in Table VI-1 to the value of the reused water. It can be seen that the net benefits are zero, that is, the system can be considered break-even in an economic sense when the value of water is approximately $0.26 per thousand gallons. When the value is set higher than this level, positive net benefits result, and when it is lower, a net cost develops which must be compared to the nonquantified secondary benefits described in later sections.

Although the conventional storm water treatment system developed by Whitman, Requardt and Associates can be said to be fairly representative of the best available method of achieving the water quality objective using current technology, the sensitivity of the conclusions to pollution control costs associated with advanced technology should be investigated. The local storage concept itself offers an advanced method of water pollution control when optimized for that purpose alone, without reuse. This type of system has been described in Chapter IV and will be compared to the conventional approach later in this chapter. Using the costs of the local storage system without reuse as the imputed value of water pollution control also permits convenient examination of the economics of reuse separately from the storm water control and treatment aspects of the system. Table VI-2 summarizes economic analyses of the potable reuse system as a multipurpose project, in each case using the costs of the local storage system without reuse as the imputed value of water pollution control. The lower value assigned to the water pollution control function results in lower net benefits in the multipurpose analysis and a higher incremental cost of treated water. The analysis of the system as a water pollution control project is, of course, unchanged. Figure VI-2 shows the effect of the value of treated water on the net benefit function when the local storage system is used as the source of water pollution control costs.

2. Sub-potable Reuse

The identification of capital costs associated with the optimum system for sub-potable reuse is described in Chapter IV. These costs, after adjustment, total $2,598,600, of which $1,562,700 represent distribution system costs. These costs are amortized over the life of the facilities and added to the operating and maintenance costs, resulting in a total system cost of $524.15 per day. Further details appear on Table VI-3. Total benefits are calculated at $463.93 per day, producing a negative net benefit or net cost of $60.22 per day. The sub-potable reuse system then, when considered as a multipurpose project, fails to generate tangible benefits equal to the costs.

Other comparisons were carried out as described in the previous section, to investigate the effect of water supply and water pollution control considerations. Table IV-3 shows the results of analysis as a water supply project and this is further illustrated by Figure VI-3, where the effect of various values of treated water is plotted. Figure VI-3 also illustrates the

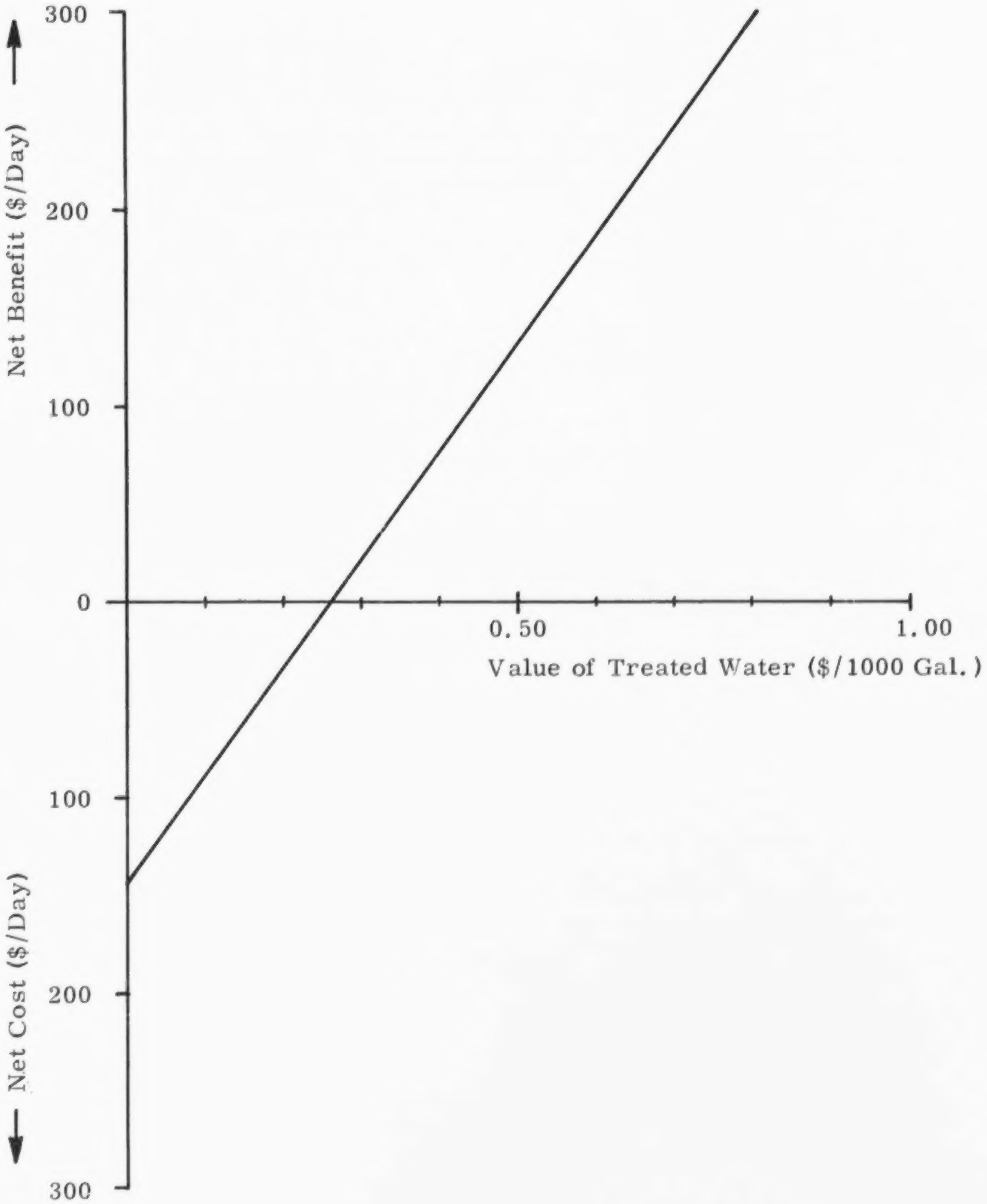

Figure V-1. Potable Reuse Net Benefits vs. Value of Treated Water - Pollution Control Costs from Conventional System

TABLE VI-2. ECONOMIC ANALYSIS OF POTABLE REUSE SYSTEM VS. LOCAL STORAGE WITHOUT REUSE

Costs	
Capital Cost of System	$ 1,445,200.
Amortized Capital Cost ($/Day)	207.21
Operating and Maintenance Cost ($/Year)	70,845.00
Operating and Maintenance Cost ($/Day)	194.10
Total System Cost ($/Day)	401.31
Benefits	
Water Reuse	
Quantity of Reused Water (Gallons/Year)	202,942,000.
Value of Reused Water ($/Year@$0.45/1000 Gallons)	91,324.
Value of Reused Water ($/Day)	250.20
Water Pollution Control	
Capital Cost of Local Storage System	829,800.
Amortized Capital Cost ($/Day)	118.99
Operating and Maintenance Cost ($/Year)	20,000.
Operating and Maintenance Cost ($/Day)	54.79
Total System Cost ($/Day)	173.78
Total Benefits ($/Day)	423.98
Multipurpose Water Resource Project	
Net Benefits ($/Day)	22.67
Benefit/Cost Ratio	1.06
Water Supply Project	
Total Cost Less Water Pollution Benefit ($/Year)	83,050.
Net Cost of Treated Water ($/1000 Gallons)	0.41
Water Pollution Control Project	
Total Cost Less Water Supply Benefit	55,160.
Net Cost of Pollution Control ($/Acre-Year)	48.05
($/DU*-Year)	14.68

*DU = Dwelling Unit

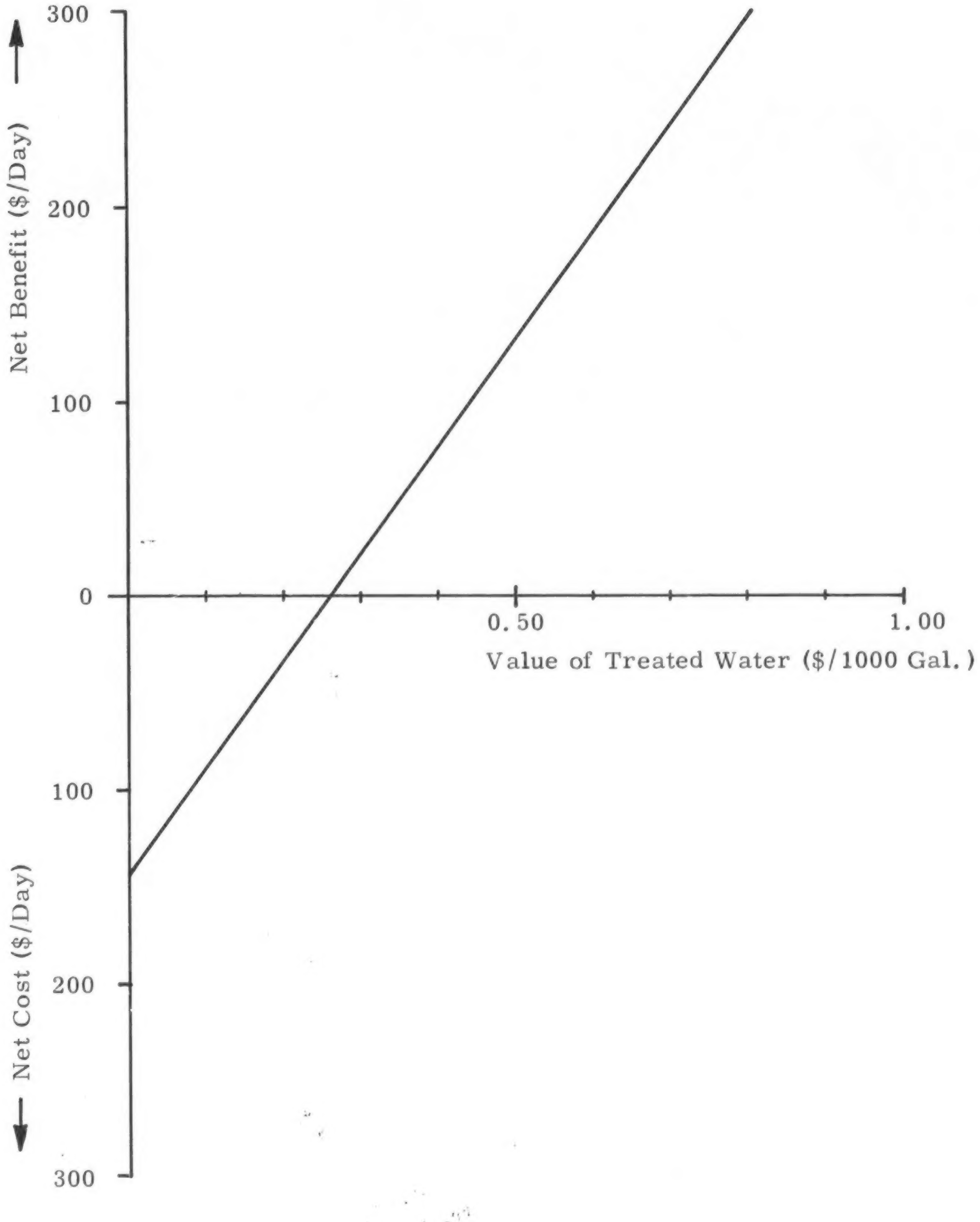

Figure V-1. Potable Reuse Net Benefits vs. Value of Treated Water - Pollution Control Costs from Conventional System

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Value of Reused Water ($/Day)	250.20
Water Pollution Control	
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Amortized Capital Cost ($/Day)	118.99
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Operating and Maintenance Cost ($/Day)	54.79
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Total Cost Less Water Supply Benefit	55,160.
Net Cost of Pollution Control ($/Acre-Year)	48.05
($/DU*-Year)	14.68

*DU = Dwelling Unit

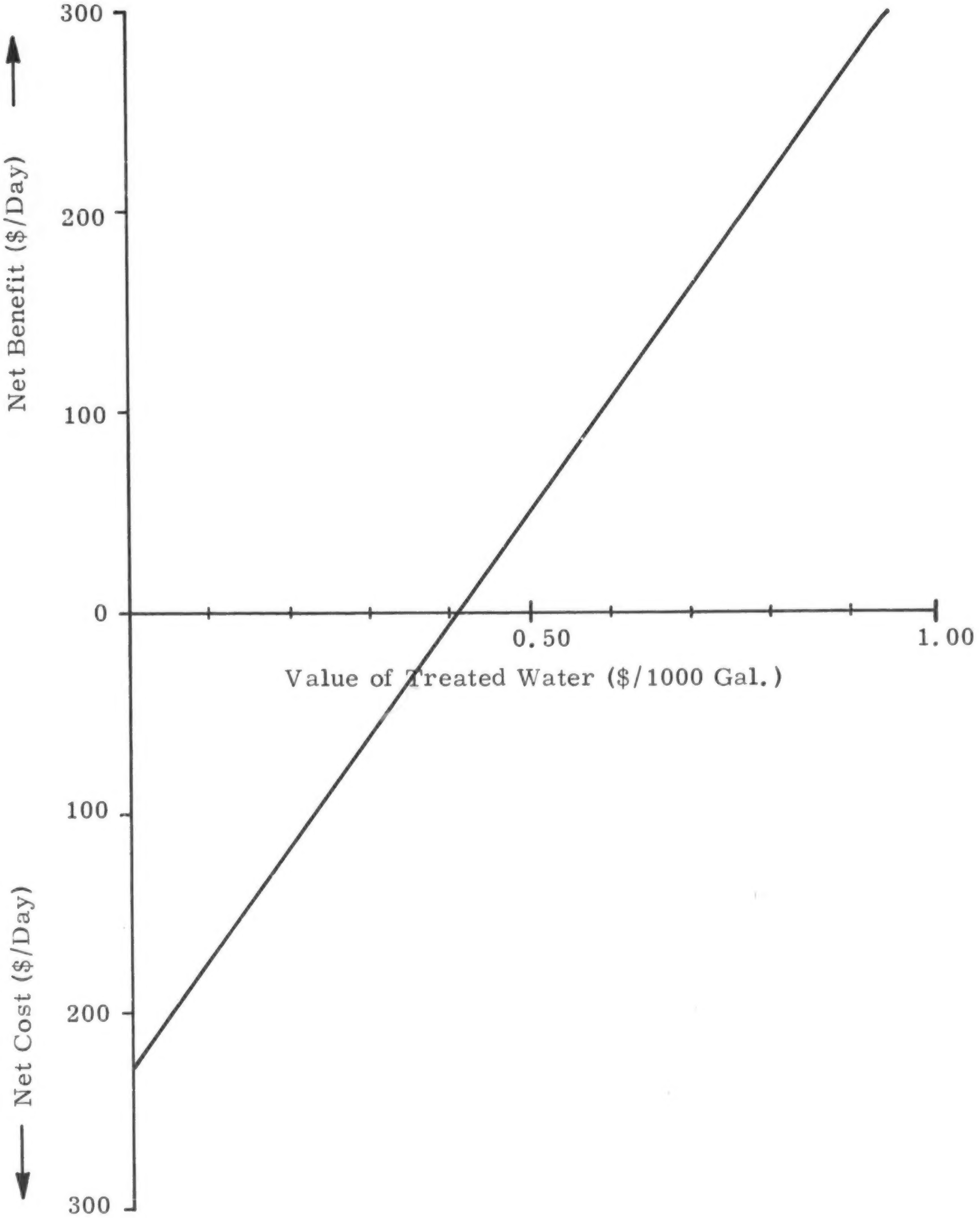


Figure VI-2. Potable Reuse Net Benefits vs. Value of Treated Water - Pollution Control Costs from Local Storage System

TABLE VI-3. ECONOMIC ANALYSIS OF SUB-POTABLE REUSE SYSTEM VS. CONVENTIONAL STORM WATER TREATMENT

Costs	
Capital Cost of System	$ 2,598,600.
Amortized Capital Costs ($/Day	372.64
Operating and Maintenance Cost ($/Year)	55,300.
Operating and Maintenance Cost ($/Day)	151.51
Total System Costs ($/Day)	524.15
Benefits	
Water Reuse	
Quantity of Reused Water (Gallons/Year)	167,849,000.
Value of Reused Water ($/Year@$0.45/1000 Gallons)	75,532.
Value of Reused Water ($/Day)	206.94
Water Pollution Control	
Capital Cost of Convetional System	1,314,500.
Amortized Capital Cost ($/Day)	188.50
Operating and Maintenance Cost ($/Year)	25,000.
Operating and Maintenance Cost ($/Day)	68.49
Total System Cost ($/Day)	256.99
Total Benefits ($/Day)	463.93
Multipurpose Water Resource Project	
Net Benefits ($/Day)	-60.22
Benefit/Cost Ratio	0.89
Water Supply Project	
Total Cost Less Water Pollution Benefit ($/Year)	97,510.
Net Cost of Treated Water ($/1000 Gallons)	0.58
Water Pollution Control Project	
Total Cost Less Water Supply Benefit ($/Year)	115,780.
Net Cost of Pollution Control ($/Acre-Year)	100.85
($/DU*-Year)	30.82

*DU = Dwelling Unit

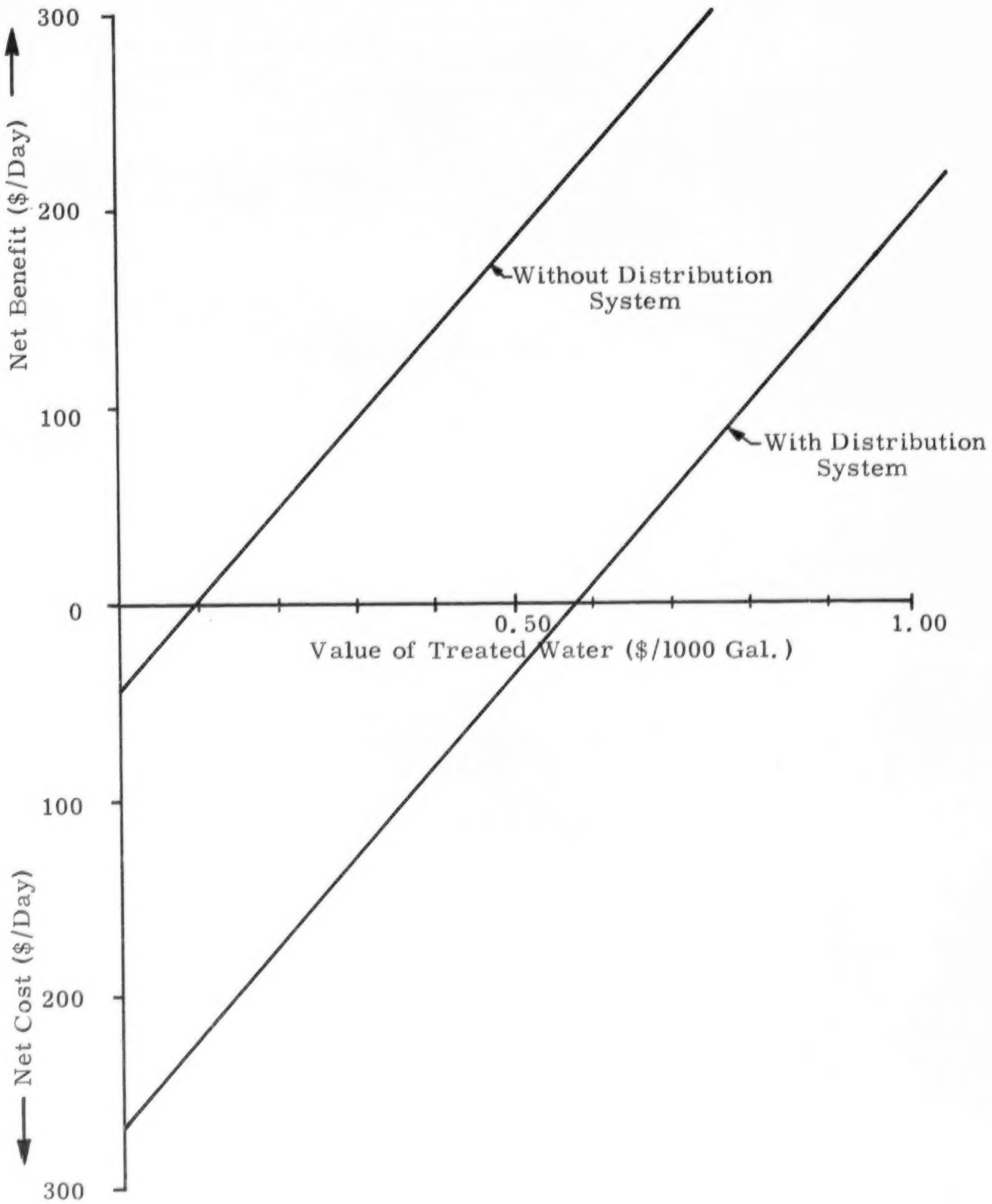


Figure VI-3. Potable Reuse Net Benefits vs. Value of Treated Water - Pollution Control Costs from Conventional System

influence of the cost of the distribution system on the overall results. It can be seen that treated water could be priced at $0.10 per thousand gallons at the treatment plant (the zero cost intercept of plot "without distribution system") but that the cost of providing the distribution system increases the break-even price to $0.58 per thousand gallons. Recomputation of benefits based on the use of the local storage system to establish the value of water pollution control leads to Table VI-4 and Figure VI-4. It can now be seen that the portion of the total system cost allocable to water supply is substantially higher, but that the apparent cost of water pollution control remains at $30.82 per dwelling unit.

3. No Reuse

Costs for the local storage system without reuse have been presented in previous sections as an alternative measure of the value of water pollution control. Total system cost has been given as $173.78 per day, compared to $256.99 per day quoted for the conventional system. In addition, the local storage system offers the secondary, unquantified benefits described later in this chapter.

4. Summary

The economic comparison of the three local storage systems and the conventional storm water treatment system has been conducted in several different ways. When the local storage systems are treated as multipurpose water resource projects, only the potable reuse system yields a benefit/cost ratio greater than unity, regardless of the base employed for evaluation of water pollution control benefits. The ratio was as high as 1.26 when the conventional system was used as the base and fell to 1.06 with the local pollution control system as the base. Investigation of the value placed on treated water revealed that all values greater than $0.26 per thousand gallons resulted in positive net benefits when the conventional system was employed as a base. Use of the local pollution control system as a base increased the break-even point to $0.41 per thousand gallons. When the system was analyzed as a water pollution control project, the net cost of pollution control amounted to $55,160 per year, or $14.68 per dwelling unit-year. This can be compared to the cost of pollution control by conventional means of $24.97 per dwelling unit-year and $16.88 per dwelling unit-year by the local storage system without reuse. Analyzed on another basis, it can be found that if the system is installed as a pollution control system and the necessary facilities for potable water reuse are added, the additional cost involved in treating and distributing the potable water will total $0.41 per thousand gallons of water distributed.

The sub-potable reuse system exhibits benefit/cost ratios as low as 0.73 and is characterized by a moderately high net cost even after full credit is taken for all quantified benefits. The cost of treated water is higher than the cost of water from the present alternative source when the system is analyzed as a water supply project. Furthermore, when analysis is based on water pollution control considerations, the cost of pollution control is higher than any of the alternatives. The excessive cost of the

TABLE VI-4. ECONOMIC ANALYSIS OF SUB-POTABLE REUSE SYSTEM VS. LOCAL STORAGE WITHOUT REUSE

Costs	
Capital Cost of System	$ 2,598,600.
Amortized Capital Cost ($/Day)	372.64
Operating and Maintenance Cost ($/Year)	55,300.
Operating and Maintenance Cost ($/Day)	151.51
Total System Cost ($/Day)	524.15
Benefits	
Water Reuse	
Quantity of Reused Water (Gallons/Year)	167,849,000.
Value of Reused Water ($/Year@$0.45/1000 Gallons)	75,532.00
Value of Reused Water ($/Day)	206.94
Water Pollution Control	
Capital Cost of System	829,800.
Amortized Capital Cost ($/Day)	118.99
Operating and Maintenance Cost ($/Year)	20,000.
Operating and Maintenance Cost ($/Day)	54.79
Total System Cost ($/Day)	173.78
Total Benefits ($/Day)	380.72
Multipurpose Water Resource Project	
Net Benefits ($/Day)	-143.43
Benefit/Cost Ratio	0.73
Water Supply Project	
Total Cost Less Water Pollution Benefit ($/Year)	127,890.
Net Cost of Treated Water ($/1000 Gallons)	0.76
Water Pollution Control Project	
Total Cost Less Water Supply Benefit ($/Year)	115,780.
Net Cost of Pollution Control ($/Acre-Year)	100.85
($/DU*-Year)	30.82

*DU = Dwelling Unit

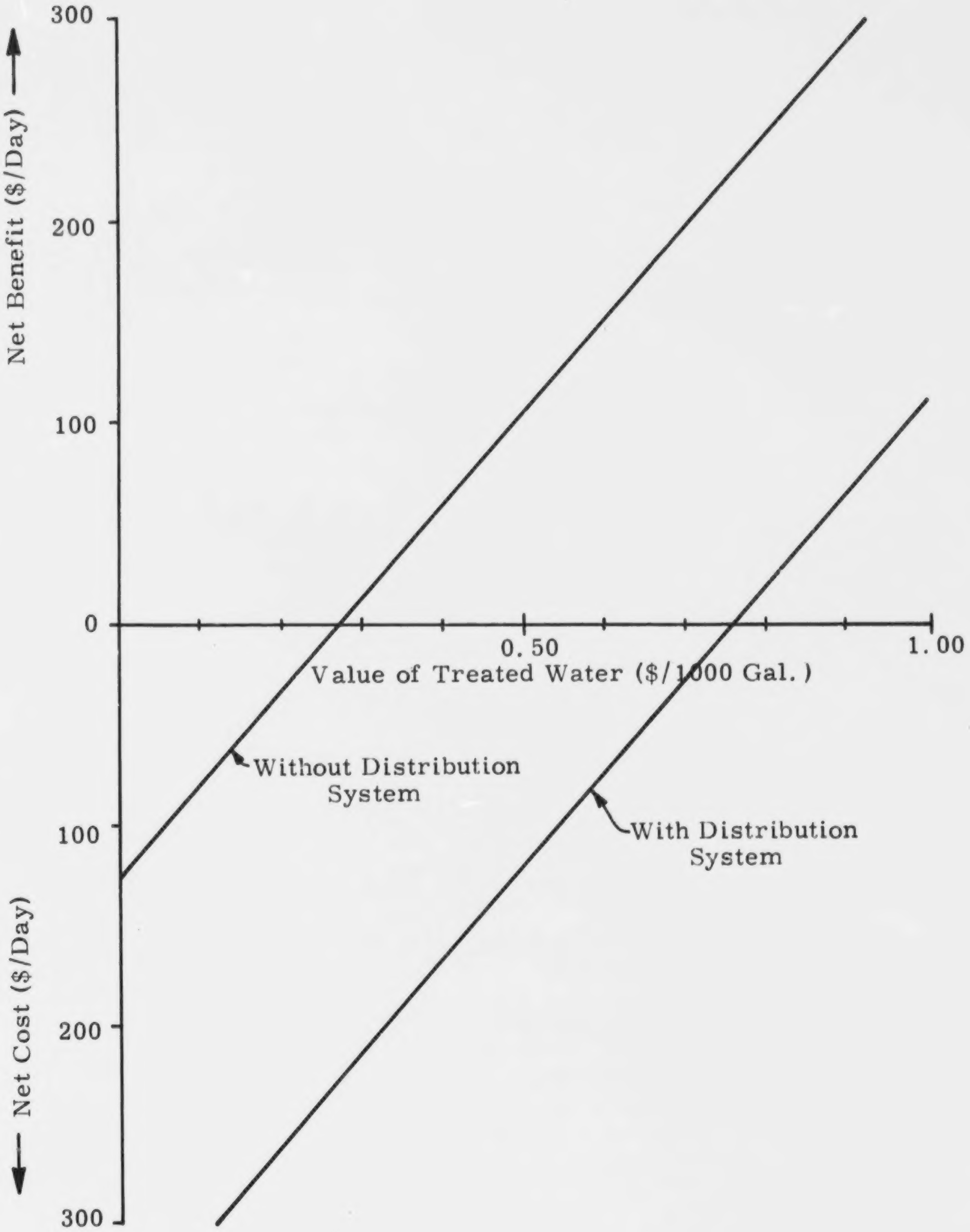


Figure VI-4. Sub-Potable Reuse Net Benefits vs. Value of Treated Water - Pollution Control Costs from Local Storage System

sub-potable system can be seen to be wholly caused by the cost of the sub-potable distribution system. The construction, operation, and maintenance of this facility alone accounts for an additional cost of approximately $0.48 per thousand gallons of treated water which is more than the total incremental cost of the alternative supply of water.

Operation of the local storage system without reuse as a water pollution control method was compared to the conventional system and found to result in a net benefit of $83.21 per day, somewhat less than the benefit associated with the operation of a potable reuse system which yields $105.88 per day.

C. Water Pollution Control

Although all systems were designed to meet the water quality standard described in Section II. D., practical considerations forced some compromise in the design of the conventional system. As noted in Chapter V, this system is expected to remove particles effectively down to 10 micron effective size during a one year storm, although removal will be more efficient during less intense storms. The local storage-pretreatment units are designed for removal of particles as small as 1.0 micron during all storms up to one year return interval intensity. An accurate appraisal on the effect of this limitation on the performance of the system would require particle size distributions for the storm flows associated with various intensity storms. Information of this type is not available for this area. It is considered that the overall quality of water discharged from the local storage-pretreatment units will be significantly better than that released from the conventional treatment basin, but full-scale tests would be required to verify this assumption. For purposes of economic comparison, it has been assumed that all systems perform equally well as water pollution control facilities. The secondary benefits described in the following sections result from the application of the local storage system and are not associated with the conventional system in the present application.

D. Water Supply

Both the potable and sub-potable reuse systems make available a water resource not now exploited in the Columbia area. The Howard County Metropolitan Commission purchases water from Baltimore City and the Washington Suburban Sanitary Commission. Both of these water utilities employ surface water sources located outside Howard County, except for portions of the Rocky Gorge and Triadelphia impoundments of the Washington Suburban Sanitary Commission, located on the Patuxent River. The portion of Columbia covered by this study lies wholly in the watershed of the Little Patuxent River which is not utilized as a water supply except by Fort George G. Meade with withdrawals of less than 6.0 million gallons daily from a point almost 15 miles downstream and below the confluence of the Little and Middle Patuxent Rivers. Full development of the potable reuse system described in this report for the Wilde Lake drainage area would result in stream flows below the Wilde Lake dam slightly greater than those typical of the period before development of Columbia began. At the same time, more than 52 percent of the total water demand of the Wilde Lake watershed would be satisfied by the potable reuse system. Application of the sub-potable reuse system would result in more than 41 percent of the total demand being satisfied from this source. Since these systems not only make a presently unused water resource available, but improve the quality and maintain the level of downstream flow, the water supply aspects of the concept have been given considerable emphasis in this study.

E. Secondary Benefits

The primary objectives and benefits of the local storage, treatment, and reuse concept have already been discussed. Although the system was designed for optimum realization of these primary benefits, other benefits, both tangible and intangible, do result from the installation of such a system in an urban watershed. Although these benefits may be considered secondary, they are nevertheless important and might be as economically significant in many watersheds as pollution abatement and reuse.

1. Flood Control

As any area begins to urbanize, the hydrology of the watershed alters drastically. With an increase in imperviousness and improved channels, the runoff from a given storm increases both in volume and peak flow. The previously derived regression model and hydrograph analysis clearly indicate these effects. Such increases generally result in greatly raising the potential for flooding the low areas adjoining the waterways. Flood plains which, in a natural state, were subjected to inundation once every few years might become flooded several times each year.

To indicate the seriousness of this problem, reference can be made to Figure II-9 which shows that a natural area which is developed to an imperviousness of 0.35 will yield 450 percent more runoff from an inch of rainfall. The effects on the peak flow may be just as serious, with 100 percent to 300 percent increases, depending upon the configuration of the drainage basin.

In areas where the flood plain is developed, flooding resulting from urbanization can cause severe economic losses. Even in those areas where the flood plain is left as dedicated open space, the use potential decreases as the probability of flooding increases.

A system of small reservoirs located along the stream reaches within the watershed, as previously described, has the ability to significantly reduce the frequency of flooding. This has been demonstrated in the past with large reservoir systems, and the principles underlying these flood control projects hold true for the local systems discussed in this report.

As a means of graphically portraying the flood control potential of these reservoirs, an analysis was made of the operation of two small storage reservoirs in series during a large storm. For simplicity, the watersheds supplying these reservoirs are hypothetical, with approximately rectangular shapes. It was assumed that initially the watersheds had an imperviousness of 0.1 and, using the hydrograph analysis and stream routing equations, the runoff hydrographs at the outflow points in each basin were calculated for a five year storm.

A development of the basins was then assumed to take place with the imperviousness of the upper basin increasing to 0.5 and that of the lower basin to 0.3. This development also altered the lag time in each basin, reducing it from 12 minutes to four minutes in the upper basin and from

24 minutes to 12 minutes in the lower basin. Two more calculations of the runoff hydrographs were performed assuming first that no reservoirs existed, and then that reservoirs were installed immediately above the outflow point in each watershed. The three basin configurations used for analysis are shown in Figure VI-5, with the size of the areas and reservoirs indicated.

It was necessary to select an initial water level in the reservoirs since their ability to dampen flood peaks is dependent upon the available storage. To accomplish this, a frequency distribution of water levels in one of the proposed Wilde Lake reservoirs was prepared. A probability level of 0.85 was chosen as a reasonable (and conservative) criteria for selection of a storage level. The frequency distribution indicated that for 85 percent of the time, during the wettest year, the reservoir was at one-third capacity or less.

Using this value for both reservoirs, hydrographs were obtained for the developed area without reservoirs, and were then routed through the reservoir storage. This yielded three different hydrographs at the two outflow points: one from the natural area, one from the developed area with no reservoirs, and one from a reservoir system.

The results of this analysis are shown in Figures VI-6 and VI-7, which are plots of the runoff hydrographs at points A and B, the outflows of the two basins. It can be seen that at point A the development increased the peak flow by 112 percent with no reservoirs and actually reduced it to a level below the natural peak with a reservoir. At point B the increase was 110 percent with no reservoir and only nine percent with the reservoir.

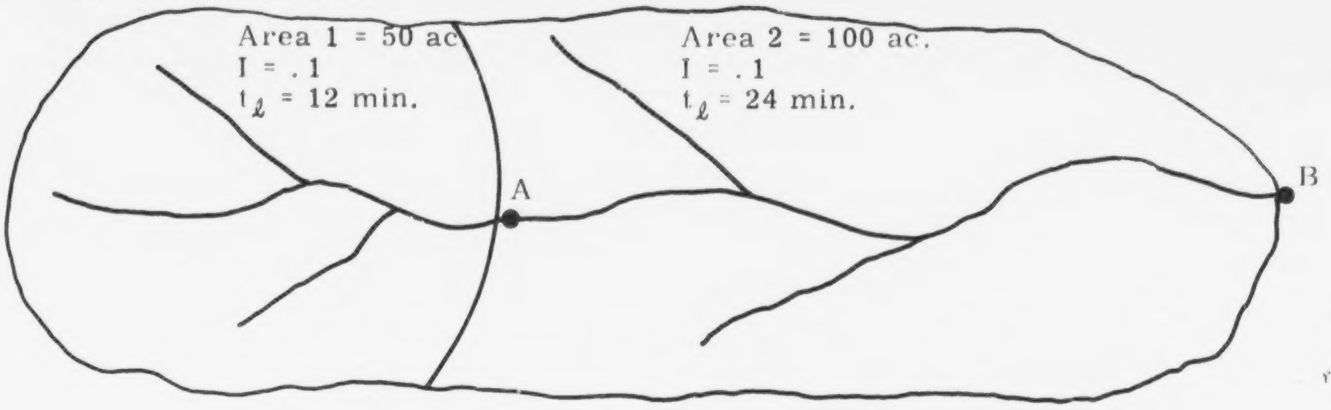
This analysis clearly indicates the ability of the local storage reservoirs to dampen the runoff peak for even very large storms. The savings in economic losses due to flooding in urban areas can be an important consideration in any analysis of the desirability of such a reservoir system.

2. Erosion Control

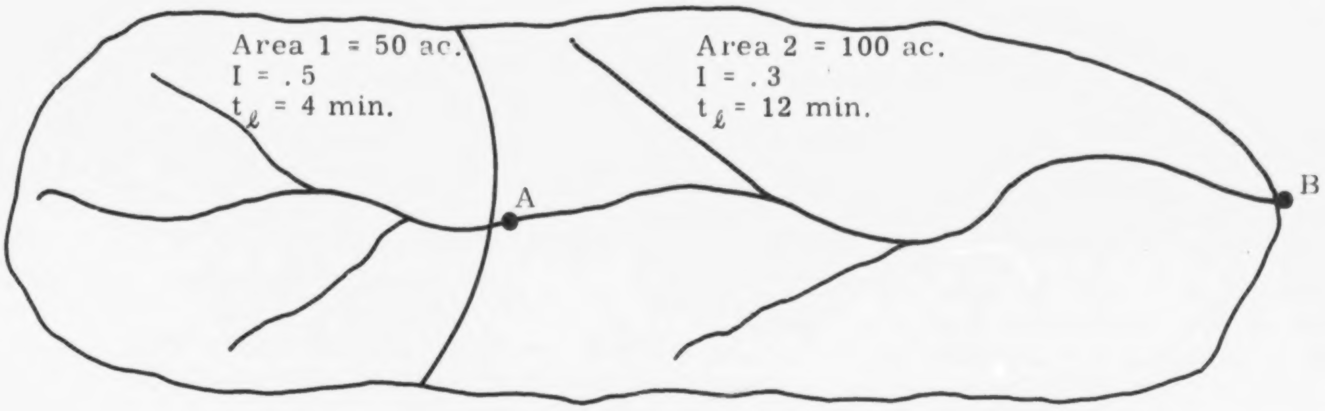
Any area, whether in a natural condition or undergoing urbanization, is subject to erosion. This process is entirely a natural one, resulting from the action of both rainfall and runoff on the ground surface and stream channels. It may, however, be tremendously accelerated by man-induced changes in the hydrology of an area. Such accelerations are well documented, with typical sediment yields from areas under construction or already developed 10 to 1000 times those from the natural area.

Although much of this increase in sediment and erosion can be attributed to the stripping of vegetation from the land during development, a still significant portion results from the increased runoff discussed previously. The erosion of either land surface or stream channels is a function of both the kinetic energy per unit volume and the total volume of the runoff. Since kinetic energy is proportional to the velocity squared, a large change in velocity or flow rate will have an even greater effect on the kinetic energy and erosion potential of the water. Velocity, flow, and volume increases are all significant during urbanization, and even after

1. Natural Area


Area 1 = 50 ac
I = .1
t_ℓ = 12 min.
Area 2 = 100 ac.
I = .1
t_ℓ = 24 min.
A
B

2. Developed Area without Reservoirs


Area 1 = 50 ac.
I = .5
t_ℓ = 4 min.
Area 2 = 100 ac.
I = .3
t_ℓ = 12 min.
A
B

3. Developed Area with Reservoirs

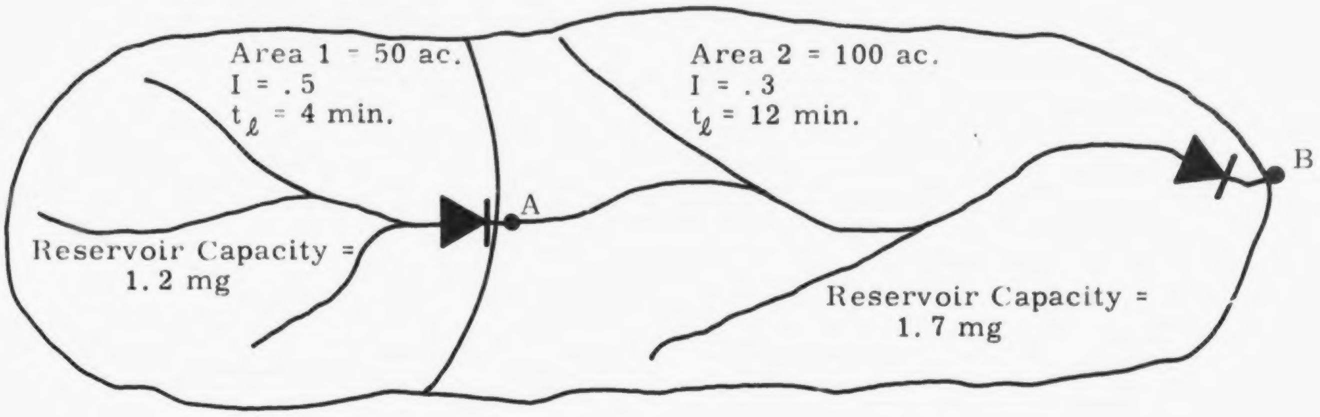

Area 1 = 50 ac.
I = .5
t_ℓ = 4 min.
Area 2 = 100 ac.
I = .3
t_ℓ = 12 min.
A
B
Reservoir Capacity = 1.2 mg
Reservoir Capacity = 1.7 mg

Figure VI-5. Basins Used for Hydrograph Damping Investigation

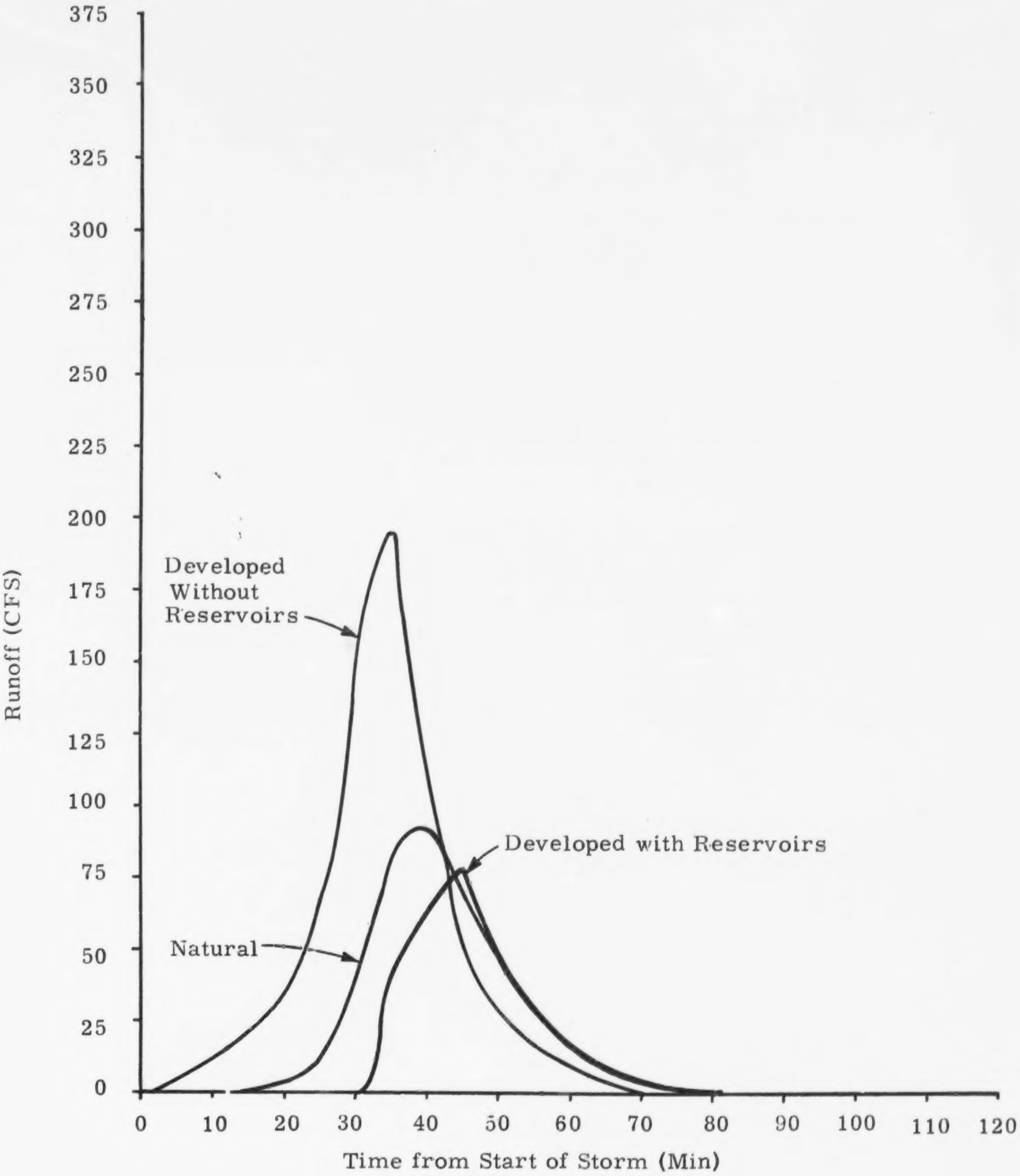


Figure VI-6. Runoff at Point A

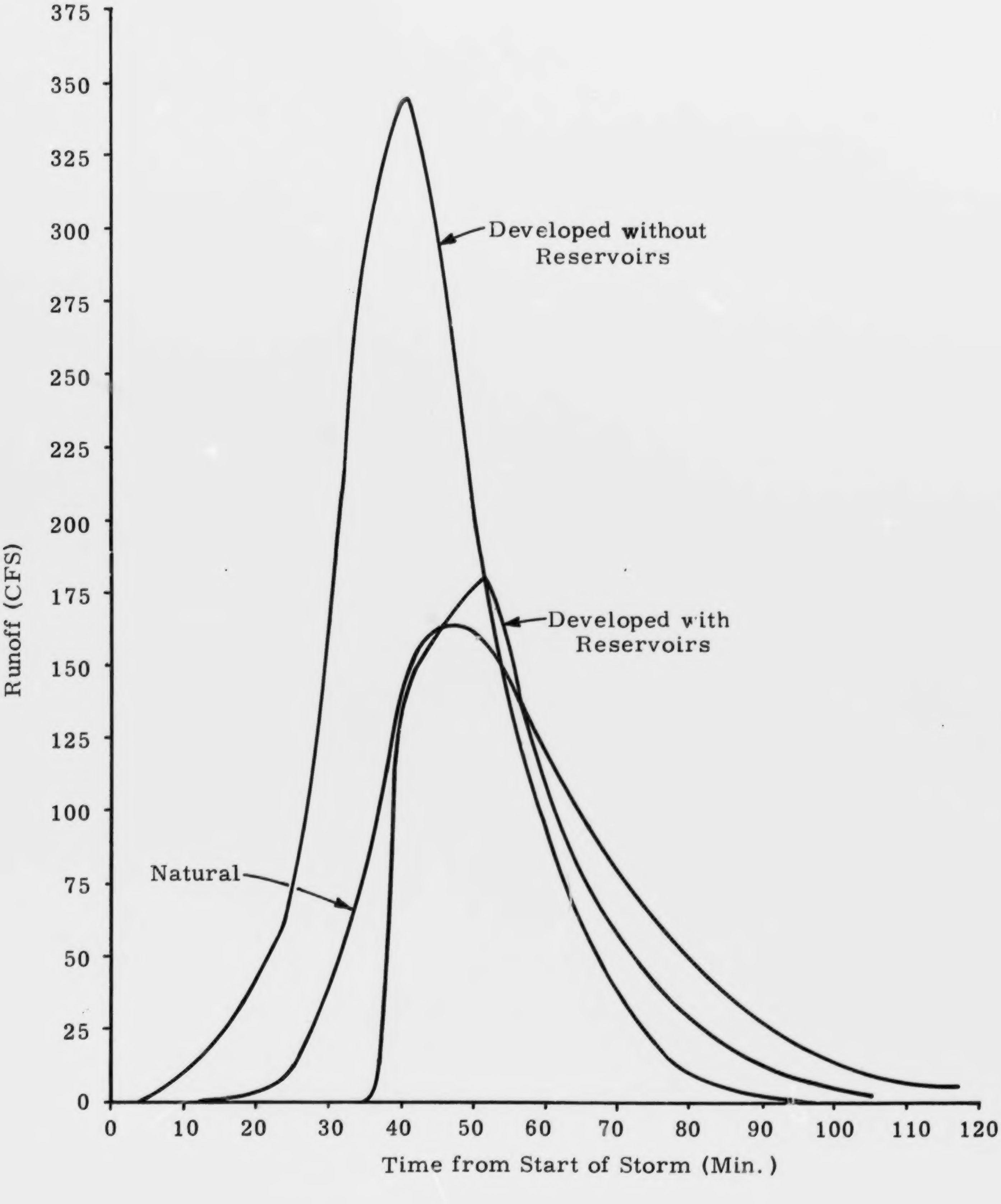


Figure VI-7. Runoff at Point B

construction in a watershed has been terminated, the increased erosion of the stream valley continues.

There are many case histories of once natural streams becoming severely eroded when their watersheds are developed. The stream bottoms are cut deeper and the banks begin to become undercut and collapse. Deep gulleys are formed in areas of the flood plain where there previously were small rivulets. Once a stream approaches this condition there is generally public pressure to have the banks and bottom paved, completely destroying its original character. Improved channels further accelerate peak flows and create hydraulic and erosion problems downstream.

For the reasons discussed with respect to flood control, this condition does not have to occur in an area with reservoir-controlled runoff. The reduction of flows and high volumes of water by reservoir storage can help to retain the natural stream channels and flood plains. Although dampening hydrographs does not always lead to a proportionate decrease in downstream erosion (due to changes in flow regimen) the ability to control release rate and the diversion of a substantial quantity of water to reuse should result in significant reduction in channel erosion.

3. Nutrient Reduction

An increasing awareness of the problems of nutrient enrichment of lakes and estuaries is developing throughout the country. Although many causes of such enrichment are offered, there is general agreement that urban runoff can add a significant amount of nitrates and phosphates to a receiving body.

The operation of the treatment plants associated with the storage reservoirs cannot be expected to remove more than a token quantity of the nutrients present. By returning a sizable portion of the runoff to the watershed through the reuse system, the nutrients will eventually be transported out of the area through the sanitary sewers.

While this does not remove the nutrients from the water, it does have two beneficial effects: it prevents a large portion of the nutrients from entering the small urban lakes which are prone to eutrophication, and it transfers them to a large treatment plant where they may be eventually removed economically.

4. Ecological Preservation

As a watershed changes from rural to urban, there is generally a drastic alteration of the ecological characteristics of the streams and their flood plains due to the increased storm flows, high silt loads, erosion of natural banks, and pollution.

A typical history of a small stream situated in a watershed undergoing urbanization might be as follows:

> During the stripping and excavating of the land, which takes place in the construction phases of development, the stream begins to become clogged with sediment. Bottom organisms found in the natural stream cannot live once the thick layer of silt has settled over the previously rocky bottom. Larger organisms, including the original fish population, which were dependent upon these organisms, begin to disappear. As construction proceeds, larger areas of the watershed become impervious and the storm flows greatly increase. The former banks of the stream fail to hold the increased flow, and mud flats are deposited along the stream valley. Erosion increases greatly and the combination of erosion and mud deposition begins to destroy the natural vegetation along the stream banks and in the flood plain. Eventually this entire area changes in appearance with little of the natural flora or fauna remaining. Trash, debris, and pollutants carried by the stream help to spawn a new ecology, usually including rats, leeches, scavenger fish, and mosquitoes. The stream valley in this condition is shunned by the residents of the watershed.

Such a history, although typical, is not necessary. As was previously discussed, a series of reservoirs within the flood plains of the streams can abate many causes of the destruction of a natural ecology. The elimination of most sediment, the dampening of floods, and the reduction of erosion can assist in the maintenance of the original ecological conditions not only in the stream but in the surrounding flood plain as well. The effects of preserving this original appearance on the recreational potential of the area are discussed in the following section.

5. Recreational and Aesthetic Considerations

Water is an important part of our natural landscape and environment. Its value has been recognized and employed by planners and administrators in projects such as the Baltimore City Inner Harbor Development and the Columbia City Project in Maryland. The aesthetic and recreational value of water, whether in a natural or urban setting, cannot be overemphasized.

In the case of the local storage ponds, the setting would be primarily a natural one with as little excavation as possible and immediate replanting of natural vegetation. The choice and treatment of construction materials would become a part of and possibly enhance the local natural environment.

The ponds present the opportunity for development of both active and passive recreation areas adjacent to the water. Aside from creating a body of water, the development of the ponds produces clear flowing streams of good quality water, thus improving the entire flood plain. With the now decreased flood activity, the flood plain can be developed and treated as useful productive land.

Three possibilities of handling the pond and its flood plain are: a completely natural setting, development of a passive recreational area, and development of an active recreational area.

In the completely natural setting, care is taken during construction to preserve the character of the area leaving as much natural vegetation and rock formations as possible. After construction, the area is landscaped to resemble the surrounding environment. The spillway is masked with rock or vegetation and the entire area is allowed to be overtaken by the wild vegetation present. Figure VI-8 shows the appearance of a typical pond in a natural setting.

The development of the flood plain as a scenic walk and picnic area with seating facilities and lighting is another possibility. Incorporating the pond, spillway, and natural surroundings into a creative, stimulating design could prove quite valuable to the local community. Such a design concept is illustrated by Figure VI-9.

Simple structures could be developed in more populated areas to turn the pond and spillway area into a recreational playground facility. In the event that local conditions dictate fencing of the ponds, the development of these recreational facilities will produce a positive result by having the fencing contain a usable function rather than merely guarding the pond. One recreational design that might be considered for such an application is shown in Figure VI-10.

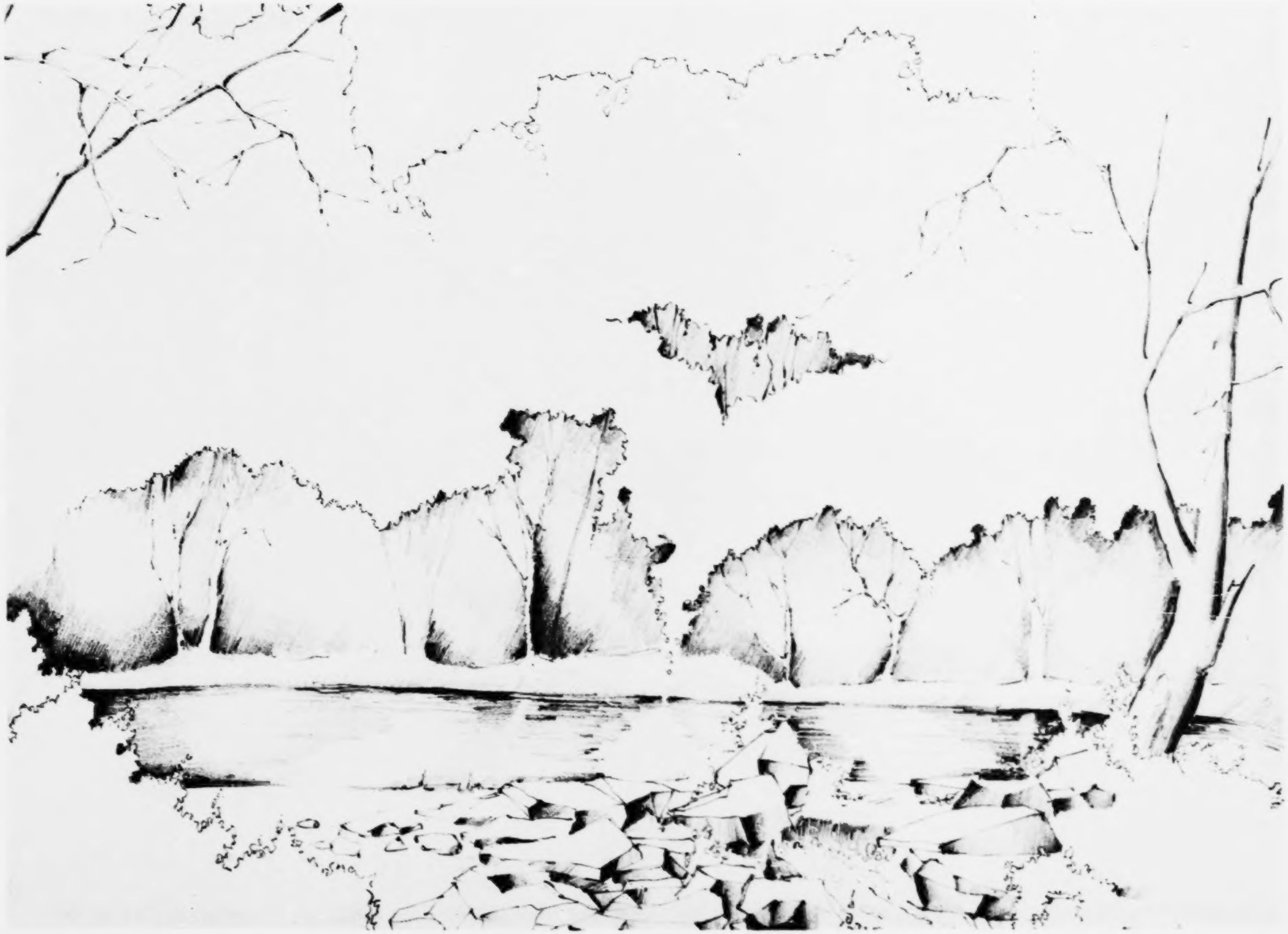


Figure VI-8. Local Storage Pond in Natural Setting

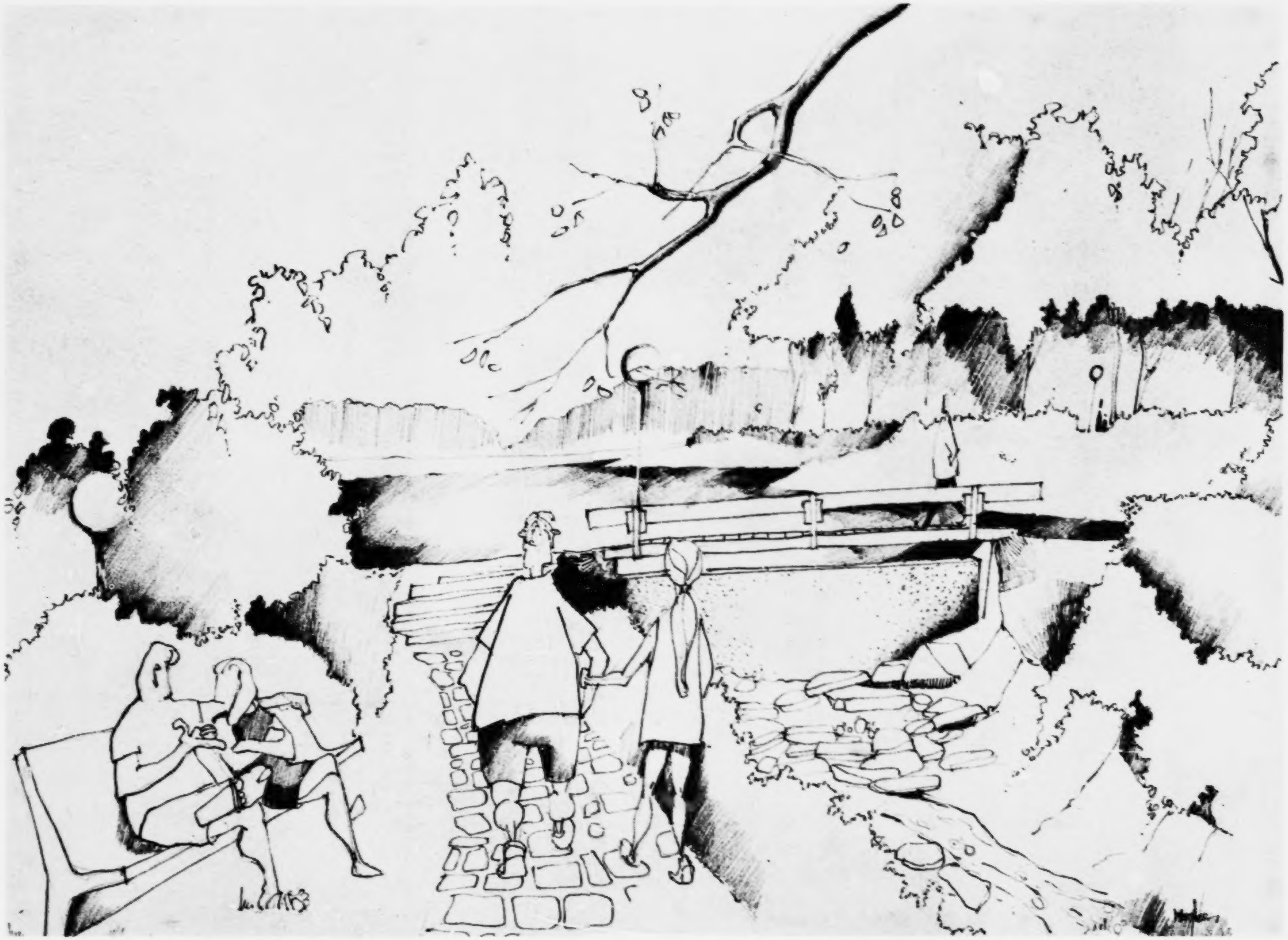


Figure VI-9. Local Storage Pond in Wooded Park Setting

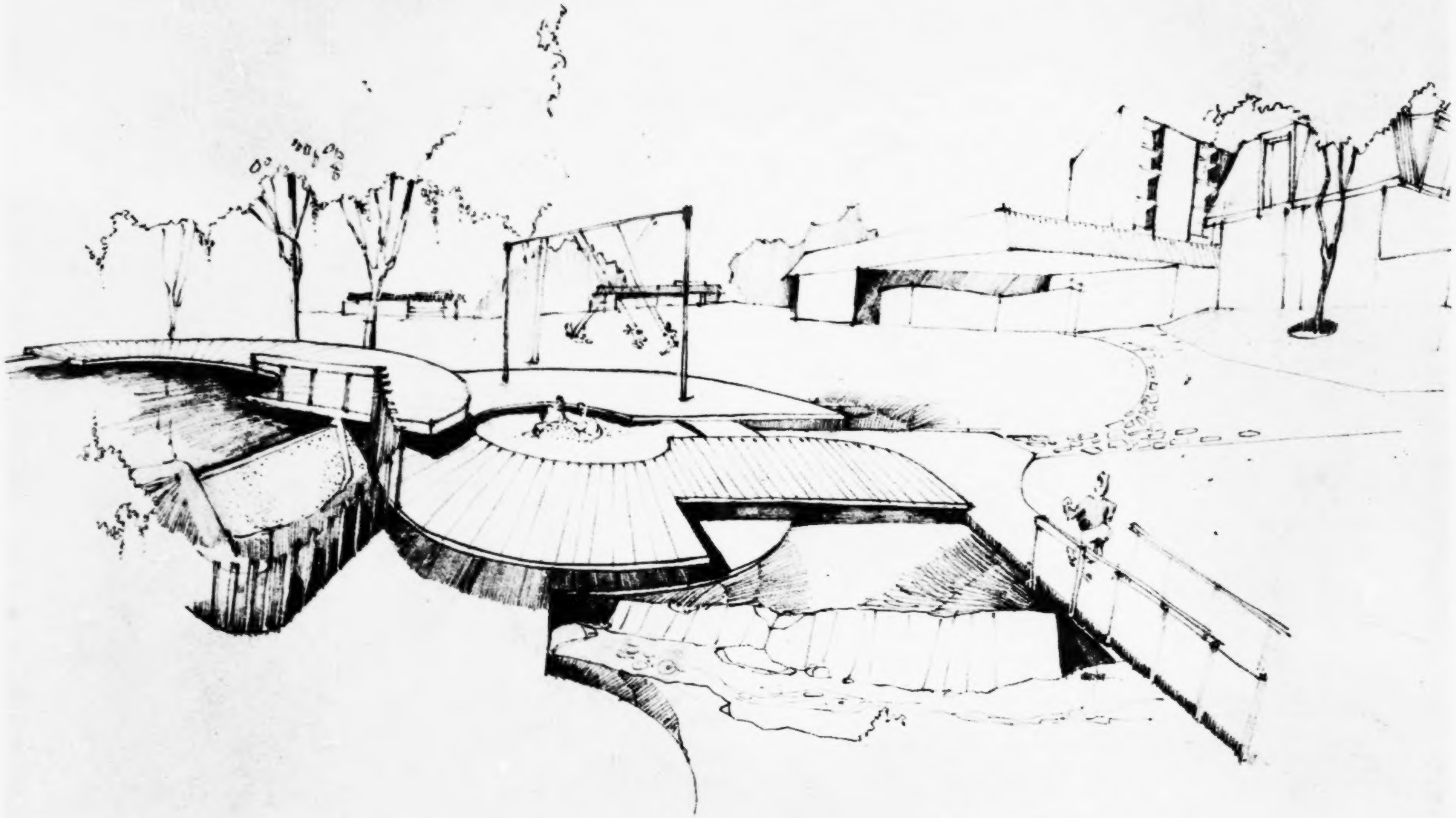


Figure VI-10. Local Storage Pond in Recreation Area Setting

VII. DEMONSTRATION PROGRAM

A. Introduction

The "Phase I - System Study, Design, and Evaluation of the Local Storage, Treatment, and Reuse of Storm Water" included the system analysis and design efforts reported in the previous chapters of this report and the planning of a demonstration program to be conducted as Phases II and III of this program. As originally envisioned, Phase II would consist of the detailed design and construction of the demonstration system and Phase III would consist of the operation, test, and evaluation of the demonstration system.

Throughout the analysis and design of the local storage system as an advanced method of storm water pollution control, the program was strongly oriented toward the objective of attaining concepts that could be physically demonstrated in Columbia, Maryland, and specifically the Wilde Lake watershed. It was also considered that the concepts to be demonstrated should provide the necessary design and economic information to permit the local storage, treatment, and reuse concept to be applied to other regions of the country.

Reviewing the results of the "Systems Evaluation" reported in Chapter VI of this report, the following are the major conclusions with respect to the technical and economic feasibility of the local storage, treatment, and reuse of storm water system concept that were considered in the formulation of the demonstration program.

1. Pollution Control

a. Advanced treatment methods are required to attain reasonably sized and economically feasible reservoirs.

b. The storage of storm water runoff represents a major problem irrespective of the type of storage employed.

c. The use of open reservoirs provides a more economical means of storage than any type of constructed storage.

d. Treatment of storm water at storm runoff flow rates is not practical and storage for subsequent treatment must be provided.

e. Sizing reservoirs to provide necessary detention for natural sedimentation of pollutants results in extremely large reservoirs.

f. An inclined tube settler, with chemical treatment, following a storm retention basin is considered to be the most attractive treatment method of all the concepts reviewed in this study.

g. The local storage system provides a lower capital cost ($829,800 versus $1,314,500) and lower operating and maintenance cost ($54.79 versus $68.49 per day) than a conventional central storage and treatment system in the case of the Wilde Lake watershed.

h. The local storage system provides greater pollution control, both in terms of percentage of total water runoff treated and the degree of treatment, than the conventional central storage and treatment system in the case of the Wilde Lake watershed.

i. The local storage system has the added advantage of damping storm water runoff flows resulting in flow rates that are comparable to the natural conditions prior to development thereby reducing stream bed erosion and the attendant pollution.

j. A local storage pollution control system would consist of a number of reservoirs and treatment facilities, operating in parallel and in series, and interactions of the reservoirs must be considered.

2. Storm Water Reuse

a. The amount of storm water available represents a substantial portion of the overall demands of the watershed and maximum utilization of this water requires that all potable and nonpotable uses be considered.

b. The sizing of local storage reservoirs as independent water supplies requires relatively large and costly reservoirs, hence storm water should be considered as only a supplemental source.

c. The optimum benefit from the reuse of storm water results when treated to drinking water standards and distributed through existing water distribution mains. In the case of the Wilde Lake watershed:

(1) The benefits from reuse could completely offset the cost of the local storage pollution control and reuse system, if reservoirs could be sized and located without land use constraints.

(2) Considering the existing land use constraints, the net cost of attaining pollution control will be substantially lower than either the local pollution control system without reuse or the conventional pollution control system. ($151.11, $173.78 versus $256.99 per day, respectively for the 1140 acre watershed.)

(3) Even with existing land constraints and system optimization based on minimum pollution control costs, the storm water reuse system can supply over 50 percent of the demands of the watershed.

d. Demonstration of the feasibility of operating and maintaining, with limited attendance, a number of small treatment plants capable of reliably producing drinking water from storm water will be required to prove the technical and economic feasibility of the optimal and least cost system of pollution control.

e. In the case of the Wilde Lake watershed, the treatment of storm water to qualities adequate for reuse in lawn watering, toilet flushing, and other sub-potable uses would:

(1) Provide sufficient water to supply over 42 percent of the demands of the watershed

(2) Provide pollution control at a net cost of $101.73 per day not including distribution or $325.83 per day including a separate distribution system, compared to $151.11 per day for a potable reuse system, $173.78 for a local pollution control system, and $256.99 for a conventional pollution control system

f. Local storage, treatment, and reuse of sub-potable water could be attractive in high water cost areas.

g. Treatment of storm water to sub-potable qualities has a greater degree of assurance than treatment to potable quality. However, feasibility of this concept will require demonstration of the acceptability of dual water systems and sub-potable reuse.

h. The local storage, treatment, and reuse system can potentially reduce nutrient pollution from storm water by (1) diverting large quantities of storm water through reuse and sanitary sewers to central treatment plants at normal flow rates and (2) moderate reduction of phosphates as part of the local treatment.

3. Program Approaches

In the planning of the demonstration program, four possible approaches were considered. These were (1) Demonstration of the Overall System, (2) Demonstration of System Components, and (3) a Combined Demonstration Project. The required facilities and associated test and evaluation programs are discussed in detail in the subsequent sections of this chapter. In summary, these four approaches are defined as follows:

a. Demonstration of the Overall System. This plan consists of the design and construction of sufficient facilities to demonstrate components of the system and the interaction of various system components. This plan would include the facilities to permit the effectiveness of the pollution control and storm water treatment and reuse to be thoroughly evaluated. From this program, it should be possible to apply the local storage, treatment, and reuse system to the complete watershed without further demonstration.

b. Demonstration of System Components. This plan is based on the demonstration of each of the key components of the local storage, treatment and resue system. From this program the necessary design data and performance characteristics of all components would be obtained. However, further demonstration would probably be required before the system could be applied to complete watershed.

c. Demonstration of System Components - Alternate Plan. This plan is based on a program in which the system aspects of pollution control would be thoroughly demonstrated and would include facilities for evaluating the effectiveness of the pollution control aspects. The storm water treatment and reuse would be evaluated using a temporary treatment installation and actual demonstration of the reuse of storm water would depend upon first demonstrating the ability to reliably treat storm water to drinking water standards.

d. Combined Demonstration Program. This plan consists of a four phase demonstration program in which pollution control is first demonstrated with a small number of storage reservoirs and pretreatment unit, then applied to the entire watershed. Similarly, treatment and resue would first be demonstrated with a single treatment plant with additional treatment plants installed after satisfactory operation was demonstrated. This plan would result in a complete system for the entire watershed.

B. Demonstration Program Objectives

The purpose of the demonstration program will be the development of sufficient design and operating information to permit the local storage, treatment, and reuse of a storm water concept to be generally applied to the solution of storm water and combined sewer pollution control problems. Accordingly, the objectives of the demonstration program will be substantially the same regardless of the approach finally taken. However, the approach will influence the degree to which objectives are attained as a function of time. The objectives for the demonstration program have been divided into two categories. The primary objectives are those major items which must be satisfactorily completed to establish the technical and economic feasibility of the local storage system. The secondary objectives will include the verification of design and analysis methods, the compilation of useful design and operating data, and the development of general technology from the demonstration project that will contribute to the urban hydrology and pollution control fields.

1. Primary Objectives

The purpose of the demonstration program will require the satisfactory attainment of the following primary objectives:

a. Demonstration of the capabilities of advanced treatment methods for treating storm water in combination with storage facilities of moderate size.

b. Demonstration of the performance characteristics of a series-parallel arrangement of local storage reservoirs for collection of storm water and control of pollution.

c. Demonstration of the ability of the local storage concept to damp storm water flows and abate stream erosion caused by increased runoff due to urbanization.

d. Demonstrate the acceptability of introducing small reservoirs into urban areas from an aesthetic standpoint.

e. Demonstrate the ability to treat reliably storm water to qualities acceptable for domestic usage using a number of small treatment plants and limited operator attendance.

f. Demonstration of the acceptability of storm water reuse by the public and local authorities.

g. Demonstration of the capabilities of using storm water as a supplemental source of water in meeting substantial portions of the demands of watersheds.

h. Definition of the construction, operating, and maintenance costs of local storage, treatment, and reuse systems and verification of economic benefits of reusing storm water to offset the costs of pollution control.

i. Demonstration of the effectiveness and impact of using local storage and treatment systems to control pollution of watersheds.

2. Secondary Objectives

The demonstration program will also provide the means of verifying design and analysis methods in actual practice, and of collecting substantial amounts of valuable data in the urban hydrology and pollution control fields. These have been designated as secondary objectives and are defined as follows:

a. Verification of the use of the hydrograph snythesis procedure for calculating the runoff flows and volumes from ungaged watersheds.

b. Verification of the use of the hydrograph synthesis procedure to determine the increased runoff flow and volume due to urban development.

c. Verification of the analytical methods used to calculate peak flows as a function of storm return intervals.

d. Collection of data on the amounts of erosion and sedimentation resulting from urbanization.

e. Collection of data changes on the type and amounts of pollutants during urban development.

f. Collection of data on pollutant content and characteristics of storm water as a function of the intensity and duration of storm water.

g. Determination of the performance characteristics and the maximum processing rate of the inclined tube settlers, as a means of controlling storm water pollution.

h. Evaluation of the effectiveness of addition of chemicals and polyelectrolytes when used in conjunction with tube settlers.

i. Comparison of the impact of storm water pollution and erosion control in areas with and without the local storage system of pollution control.

j. Verification of the methods used in the design and sizing of individual reservoirs.

k. Verification of the methods used in the design and sizing of a series of interrelated reservoirs.

l. Determination of the performance characteristics of various treatment systems in the treatment of storm water to various quality levels.

m. Determination of the problems involved in the design, construction, operation, maintenance, and control of reuse systems.

n. Collection of data on internal domestic usage of water.

o. Establishment of the cost of constructing, operating, and maintaining pollution control and treatment systems under various conditions.

p. Evaluation of the potential capabilities of the local storage treatment system in the control of nutrient pollution.

C. Demonstration of the Overall System

1. General

This approach is based on the demonstration of all of the components and systems that would be involved in the local storage, treatment, and reuse of storm water systems both as an advanced method of pollution control and as a supplementary water source. Under this approach, integrated systems would be demonstrated on a sufficient scale to permit extrapolation to full scale systems in the future. This approach assumes the work would be performed as a demonstration project with community and local governmental participation limited to approval on the use of land and concurrence on the design of the systems. This will provide the necessary design and operating data on the system to permit general application of local storage, treatment, and reuse systems at the earliest possible dates.

2. System Description

The physical facilities for the "Demonstration of the Overall System" would be located in the southwest quadrant of the Wilde Lake watershed, as shown on Figure VII-1, and would consist of the following:

a. Three open storage reservoirs at Locations A, C, and D having capacities of approximately 1,730,000 gallons, 875,000 gallons, and 1,835,000 gallons, respectively.

b. Pre-treatment units at each of the three storage reservoirs consisting of inclined tube settlers, chemical addition and chlorination equipment, and equipment for sampling and measurement of flows.

c. One storm water reuse treatment plant at Location A, with a capacity of 30,000 gpd equiped to treat storm water to various qualities, measures flows and monitor quality.

d. A dual distribution system to 100 apartments at Location B and internal piping to permit supply of toilet flushing and lawn watering demands.

e. Two gaging and sampling stations at Locations E and F.

A detailed description of this "Demonstration System" with preliminary design drawings is included in Appendix F.

The site and configuration of demonstration projects under the "Demonstration of the Overall System" approach was based on several important and interrelated considerations. These are:

a. The southwest quadrant of the Wilde Lake watershed is the least developed at the present time and construction of the demonstration project can be integrated with the other construction of the area.

b. Three reservoirs located in a "Y" arrangement are the minimum number that can be used to demonstrate the operation of a system of series-parallel reservoirs such as would be required in the application of the local storage system of pollution control to an entire watershed.

c. The area covered by three reservoirs is 279 acres, which is considered to be of sufficient scale to allow extrapolation to the complete watershed as a next step.

d. The northwest quadrant contains a sub-watershed of a comparable area. With sampling and gaging stations located in each of the two streams draining these sub-watersheds, it will be possible to obtain a comparison of a controlled and uncontrolled area for evaluation of the effectiveness and impact of the local storage treatment system.

e. The single storm water reuse treatment plant located at the reservoir at Location A will contain the necessary equipment for treating storm water to all qualities, including drinking water standards. This will permit the technical and economic feasibility of the various types of storm water treatment plants to be evaluated in a single facility.

f. The reservoir and treatment facility at Location A, is adjacent to a group of six apartment houses at Location B. These apartments are to be constructed by the developers of Columbia, Maryland, in the October 1968 - January 1969 time frame. This will permit demonstration of storm water reuse to be accomplished under conditions acceptable to the State Health Department in which possible interconnections can be controlled. Also, the schedule for the construction and occupancy of the apartments is compatible with the schedule for the demonstration system.

g. The apartment development consists of six buildings containing 200 individual units. Special meters will be installed in all buildings and the dual distribution system will be included in three of the buildings. The three buildings not having reuse will serve as a control for the evaluation of reuse.

3. Construction Plan

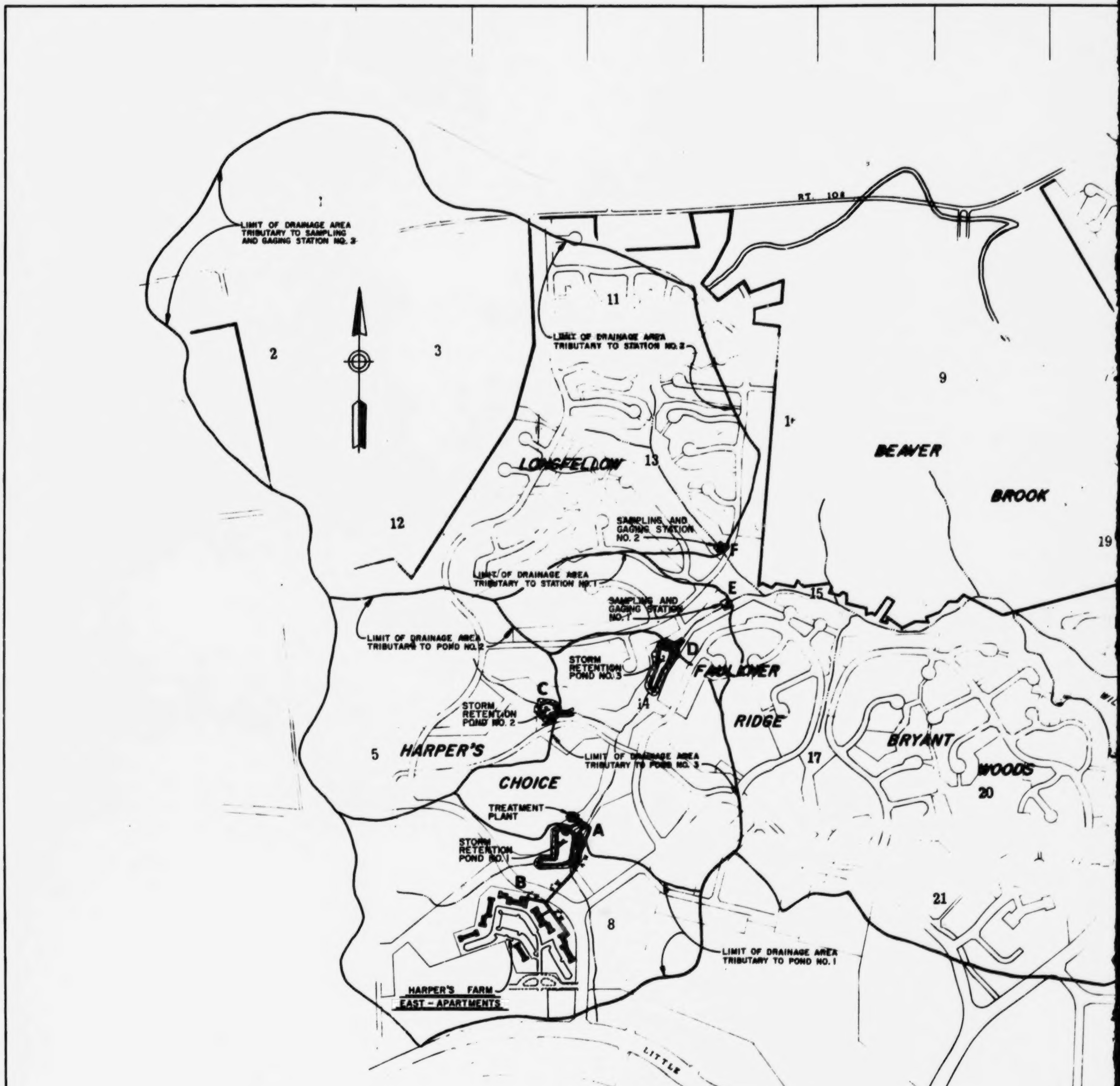
The plan for the construction of the facilities required for the "Demonstration of the Overall System" approach is based on a two-increment construction program. The items to be constructed in each increment and the schedule for design and construction is shown on Figure VII-2.

The first increment of the construction would include the construction of the two gaging and sampling stations, the reservoir and pretreatment unit at Location C and the dual distribution system in the three apartment buildings at Location B. The basis for the phasing of the construction is as follows:

a. The design and operation of the local storage and treatment system as a pollution control system is strongly dependent upon the performance characteristics of the pretreatment system. The pretreatment system consists of an inclined tube settler, chemical additions, and chlorination equipment. The need to use advanced sedimentation control systems in the control of storm water pollution in order to reduce the size of reservoirs has been discussed earlier in this report. Although available data indicates that the inclined tube settler will perform adequately with storm water runoff, this device has not been demonstrated in this type of application. Accordingly, the construction plan is based on constructing one reservoir and pretreatment unit early in the first increment in order to obtain performance data prior to commiting the construction of the remaining two reservoirs. The schedule has been planned to permit the designs of the reservoirs constructed in the second increment to be modified.

b. Both of the gaging and sampling stations would be constructed in the first increment. This will permit baseline data to be gathered on the watershed on which pollution control will be provided. This will permit an initial comparison to be made of the controlled and uncontrolled watersheds and will permit detailed evaluation of the effectiveness and impact of the local storage and treatment system as an advanced pollution control system.

c. The dual distribution system for storm water reuse must be installed in the first phase to be compatible with the schedule of the developer for the construction of this group of apartment buildings.


LIMIT OF DRAINAGE AREA TRIBUTARY TO SAMPLING AND GAGING STATION NO. 3
RT. 108
1
2
3
11
LIMIT OF DRAINAGE AREA TRIBUTARY TO STATION NO. 2
9
BEAVER
BROOK
LONGFELLOW
13
12
SAMPLING AND GAGING STATION NO. 2
F
19
LIMIT OF DRAINAGE AREA TRIBUTARY TO STATION NO. 1
E
15
SAMPLING AND GAGING STATION NO. 1
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 2
D
STORM RETENTION POND NO. 3
FAULKNER
C
14
STORM RETENTION POND NO. 2
RIDGE
5
HARPER'S
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 3
17
BRYANT
WOODS
20
CHOICE
TREATMENT PLANT
A
STORM RETENTION POND NO. 1
B
8
21
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 1
HARPER'S FARM
EAST - APARTMENTS
LITTLE PATUXENT PARKWAY
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833
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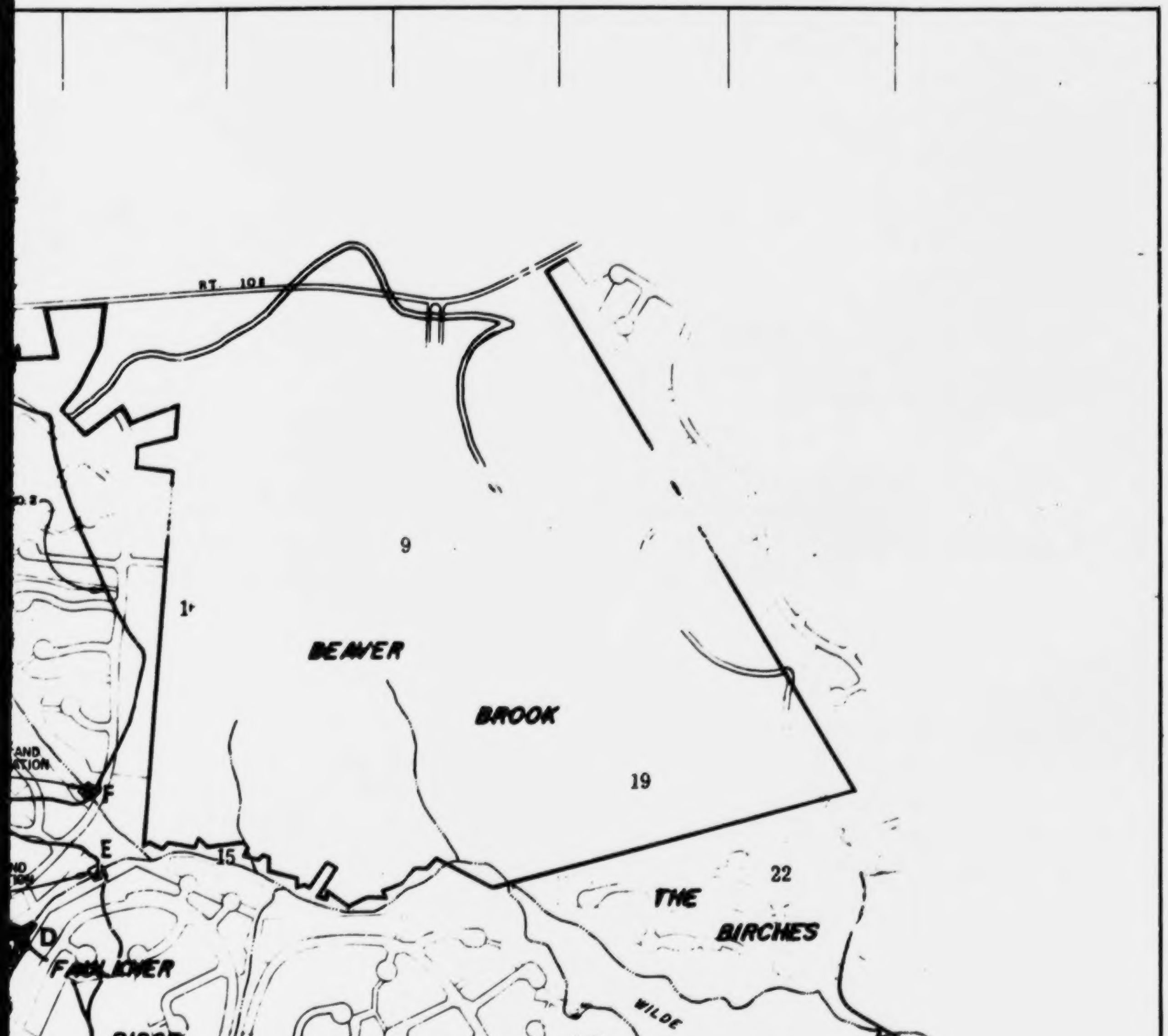
WHITMAN, REQUARDT & ASSOCIATES
ENGINEERS
BALTIMORE, MARYLAND

W.O. 5938

A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LIMIT OF DRAINAGE AREA
TRIBUTARY TO POND NO. 1
PATUXENT PARKWAY
FWPCA Contract No.14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
LOCATION PLAN
FOR
DEMONSTRATION PROGRAM
SCALE: 1" = 400'
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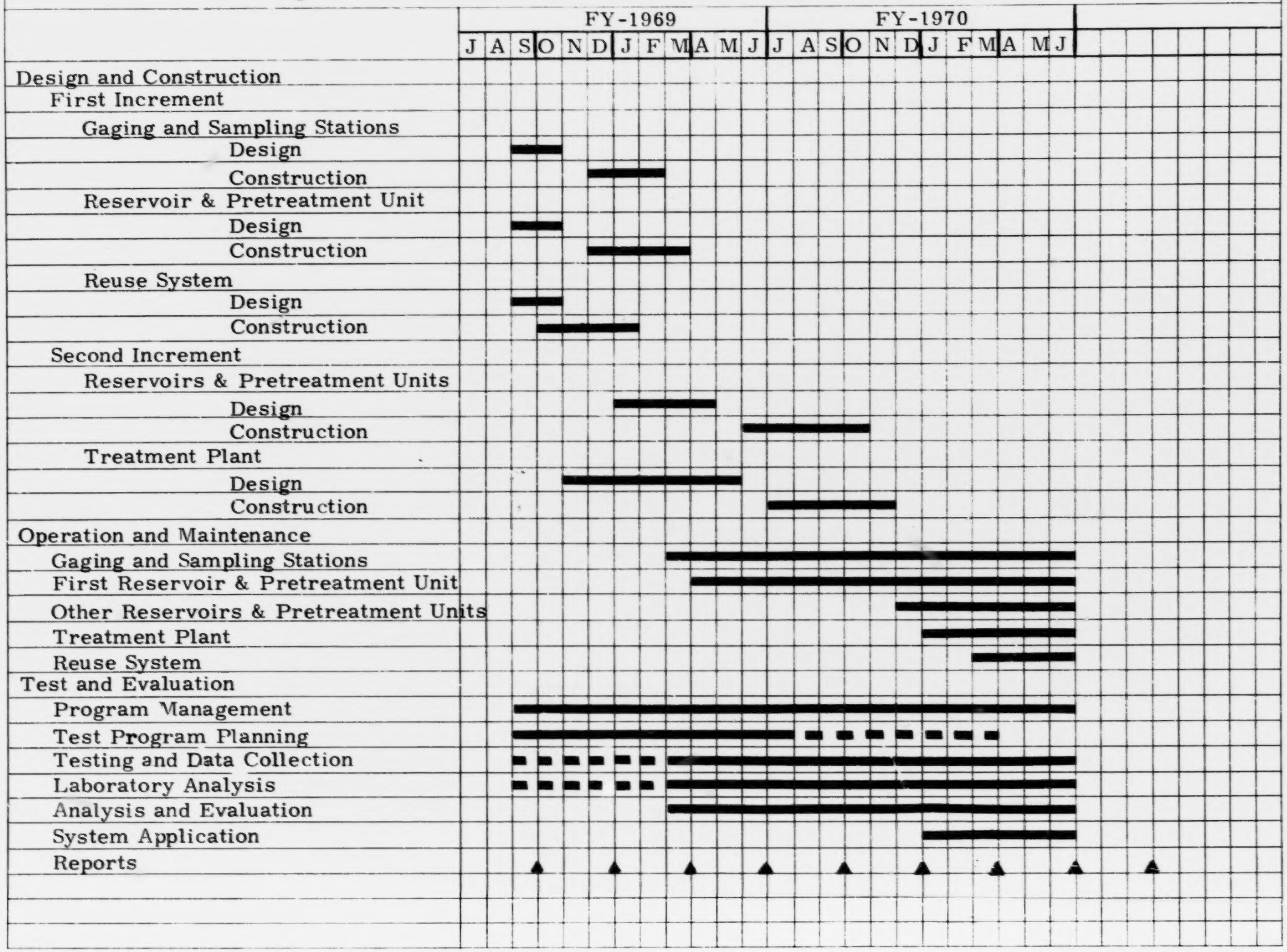
Figure VII-1. Location of Demonstration Program Facilities

B

Figure VII-2. "Schedule for the Demonstration of Overall System"



The detailed design of the facilities to be constructed in the second phase would be performed in parallel with the construction of the first phase facilities of the demonstration program. As indicated in Figure VII-2, it is planned that all of the construction will be completed in approximately one year, leaving one full year for the test and evaluation of the demonstration system based on an overall program of two years.

4. Operation and Maintenance

The operation and maintenance under the "Demonstration of the Overall System" would involve 16 months operation of the two gaging and sampling stations, 15 months operation of the first reservoir and pretreatment system and six months of operation of the treatment facility and reuse systems. In the case of the two gaging and sampling stations, this would include the changing of charts, maintenance of equipment, and the removal of sediment on a periodic basis. For the reservoirs and pretreatment units, the operational requirements would consist primarily of periodic checks and adjustments of equipment. The maintenance would consist of check and repair of equipment and the removal of sediment from reservoirs and settlers. Additional operations would be required during systems tests; however, they would be for short periods of time. The treatment plant when operating as a reuse system, would require daily attendance, but it is considered that this could be limited to the day shift with maintenance personnel available for "off shift" problems. The reuse system would not require attendance and maintenance would be minimal. The major items would be the connections required to place the system into operation and restore the system to normal operation following the completion of the demonstration program.

5. Test and Evaluation

The "Test and Evaluation" portion would overlap the design and construction of the facilities. In parallel with the design and construction of the facilities, a detailed test and evaluation program would be prepared. The "Test and Evaluation Program" would also include the management and technical direction of the design and construction of the facilities in order to assure proper coordination and the attainment of the demonstration program objectives. During this initial period, the requirements for special instrumentation would be defined and the equipment would be procured. The actual test and evaluation program would start upon the completion of the two gaging and sampling stations and the first reservoir and pretreatment system.

The following are summaries of the tasks that would be performed as part of the "Test and Evaluation" portion of the demonstration program.

a. Program Management. This task covers the effort required for management, technical direction and coordination of the detailed design, construction, operation, and maintenance, and test and evaluation of the program for "Demonstration of the Overall System." This would include the development of detailed design criteria, overall direction of the detailed design subcontracts work, review and approval of design drawings and specification, award of construction contracts, monitoring of construction, approval of field changes, and final acceptance. It would also include coordination of the program with the community and governmental officials and the management and technical direction of all the operations, maintenance, test and evaluation activities, and the preparation of monthly progress, technical status, and final reports.

b. Test Program Planning and Coordination. This task covers the preparation of an overall test and evaluation program and schedule, detailed test specifications and procedures, data collection sheets and forms for the collection of operating and maintenance data. This task also includes determination of the requirements for special instrumentation and the procurement and supervision of the installation of such equipment and provision for modifications found desirable in the test and evaluation programs.

c. Testing and Data Collection. This task covers the conduct and supervision of the specific tests and the overall data collection program. It includes the recording and compilation of test data and the initial evaluation of test results. This task would also include the collection of data on the frequency, intensity, and duration of rainfall; storm water runoff as a function of duration and intensity of rainfall and state of development of the terrain; pollutant content as a function of season and development; and other useful data on the general hydrology of the area.

d. Laboratory Analysis. This task covers the physical and chemical analysis work that would be performed in laboratories in support of the test, evaluation, and demonstration program. These would be primarily the laboratory analysis of samples collected as part of the operation of the facilities and the test and evaluation program.

e. Analysis and Evaluation. This task would cover the effort required to analyze the operation, test, and evaluation of the demonstration system. This would include analysis of construction, operating and maintenance costs and comparison with original design assumptions and estimates, performance of the demonstration project, effectiveness of various treatment methods, availability and reliability of the system, and effectiveness of the overall system as a means of pollution control and augmentation of water supplies.

f. System Development and Application. This task would provide for the development of criteria and guidelines to permit the local storage system to be evaluated for other applications. The methods used in the original design and analysis will be modified based on the results of the demonstration program and reported in a form to facilitate use in the design of future systems. Information on the construction, operating and maintenance costs of the system will be compiled with data on the effectiveness of the local storage system as a means of pollution control and as a supplemental source of water.

g. Reports. This task will cover the preparation and printing of monthly status reports, quarterly technical progress reports, and the final report.

Detail listings of the items of work to be performed under each of these tasks for the four approaches for the demonstration program are contained in Appendix G.

6. Disposition of the Demonstration System

Since the facilities constructed for the "Demonstration of the Overall System" would provide pollution control for less than a third of the Wilde Lake watershed, the future value to the community would be limited. However, it could provide the first increment for the construction of a pollution control system for the watershed. This possibility is further discussed in Section VII-E, "Combined Demonstration Program." Without expanding to a complete system, there are several possibilities for disposition of the facilities that would not involve dismantling and restoration of the site. These are:

a. Storage Reservoir and Treatment Plant. The local storage reservoir and pretreatment system and the associated reuse treatment demonstration facility may be transferred to the community for future operation and maintenance. Assuming that the technical and economic feasibility of reliably providing potable water of drinking water standards, the treatment plant could be connected to the existing water system as a supplemental source of water. Since the value of the water will probably greatly exceed the cost of operation and maintenance, continued operation by the community could be very attractive.

b. Storage Reservoirs Without Treatment Plants. Even though these two reservoirs will not provide a major benefit in the control of the pollution of Wilde Lake, they will serve as a means to dampen storm water flows and abatement of erosion in the stream channels. The community might also desire to install treatment plants at each of these reservoirs and provide additional sources of potable water.

c. Gaging and Sampling Stations. There is considerable interest in urban hydrology and the effect of urbanization on the environment. Columbia is being considered as a study area for several such studies. At present, the University of Maryland has installed rain gages and is studying the effect of urbanization upon the meteorology of the area. The capability also to measure storm water flows and pollutant content could be an important adjunct to these studies and the two gaging and sampling stations could be transferred to a University or governmental group engaged in urban research for future operation and maintenance.

D. Demonstration of System Components

1. General

This approach is based on the demonstration of those subsystems and components of the local storage, treatment, and reuse system that are most critical to the performance and economics of the overall system. This approach also assumes that the work would be performed as a demonstration project, but with a substantially reduced scope compared to the "Demonstration of the Overall System" described in Section VII-C. This approach would not include evaluation of the interaction of the reservoirs to demonstrate an overall system. Since the scale of this demonstration would be much smaller, the comparison of a controlled and uncontrolled watershed to evaluate the effectiveness and impact of the local storage, treatment, and reuse would be eliminated. Demonstration of the treatment of storm water for reuse would be included. However, the installation of a dual distribution system and actual demonstration of reuse could be eliminated to reduce the overall cost of the demonstration program. The next step in the development of the local storage, treatment, and reuse of storm water system concept would depend upon the results attained. Some for of overall system demonstration would probably be required before the local storage and treatment system could be generally applied to the solution of storm water and combined sewer pollution problems.

2. System Description

The physical facilities for the "Demonstration of System Components," would consist of the local storage reservoir at Location A on Figure VII-1 and a packaged water treatment unit at the same location. These facilities are further described as follows:

a. Local Storage Reservoir and Pretreatment Unit. This reservoir and pretreatment system would be of the same design as the one previously described for Location A, in Section VII-C-2, "Demonstration of the Overall System," and Appendix E. Under the "Demonstration of Systems Components" approach, the gaging and sampling station would be eliminated. Accordingly, instrumentation would be provided at the storage reservoir to record the runoff flow into the reservoir and the flows from the reservoir. Sampling would be accomplished using portable equipment and would include samples of water entering and leaving the reservoir for obtaining data for evaluation of the effectiveness of the local storage treatment components as a means of pollution control.

b. Packaged Water Treatment Plant. The package treatment plant will be of the design available from various manufacturers for treatment of water for small communities, camps, and industrail plants. This will be a skid mounted plant of approximately 10 gallon per minute capacity. Information on one such plant is contained in Appendix H. This plant contains provisions for chemical feed, flocculation, a settling chamber, and a mixed media filter bed. Necessary pumps and control equipment are included as part of the unit. This would also require a separate tank for backwash and clearwell storage. The package treatment plant finally selected would consider the maximum flexibility for evaluation of various treatment methods, automation for limited operator attendance and adaptability for future storm water treatment applications. This plant would be installed on a slab at approximately the same location as the treatment plant shown in Appendix F. Redwood or other patio type fencing would be used to provide an aesthetically pleasing installation.

c. Storm Water Reuse Facilities. This would consist of the dual piping system to approximately 100 apartment units located with three of the six apartment buildings adjoint to the reservoir site. The dual distribution system would be of the design described in Appendix E. For the demonstration of storm water reuse, a high outlet pressure pump would be used to pump water from the treatment plant clearwell to a hydropneumatic tank connected to the dual distribution system. Connections would be provided to permit the dual distribution system to be connected to the existing public water system during periods when treatment is not being demonstrated in order to reduce operating and maintenance costs.

3. Construction Plan

The construction of the facilities for the "Demonstration of System Components" would be divided into three separate increments. They are defined as follows:

a. Arrangements would be immediately made to have the developer modify the design of the internal plumbing in the apartments in order that the installation of the piping could be accomplished as part of the initial construction.

b. Specifications would be prepared for the packaged treatment plant and technical and cost proposals would be solicited. Selection of a treatment plant would be made in parallel with the detailed design of the facilities in order to permit the requirements to be incorporated into the design drawings for award of a single construction contract. Installation and initial checkout of the treatment plant would be included as part of the contract for the plant.

c. Detailed design drawings and specifications would be prepared for the storage reservoir, pretreatment unit, and the site facilities for the package treatment plant and reuse water pumping system. Bids will be requested and a single construction contract will be awarded for all the facilities.

Figure VII-3 contains a schedule for the design and construction of these facilities and the conduct of associated operation, test, and evaluation program.

4. Operation and Maintenance

The operation and maintenance on the "Demonstration of System Components" will include; one year of operation of the local storage reservoir and pretreatment, nine months operation of the packaged treatment plant, and a six month demonstration of storm water treatment and actual reuse. As noted in the system description, the treatment plant would be arranged to permit water from the public water system to be used to eliminate the need for continuous operation of the water treatment facility except for the designated test and evaluation periods.

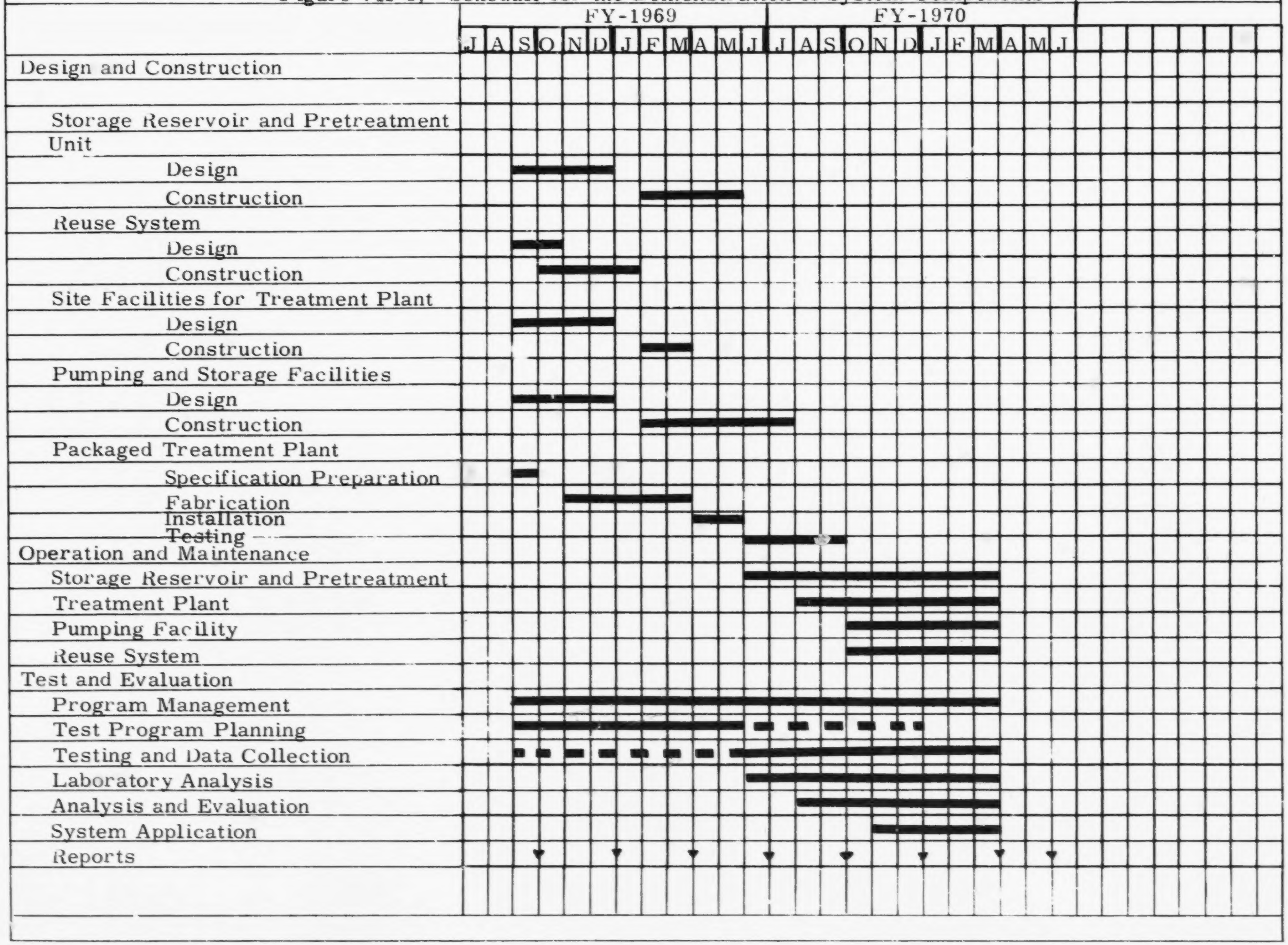
5. Test and Evaluation

The test and evaluation program would consist of the same overall tasks as were previously described in Section VII-C-4, "Demonstration of the Overall System." The scope of the test and evaluation program would be materially reduced due to the reduction of the facilities. The items of work to be performed during the test and evaluation program are listed in Appendix G.

6. Disposition of the Facilities

The disposition of the facilities would depend upon the results obtained

Figure VII-3, "Schedule for the Demonstration of System Components"



from the "Demonstration of System Components." These facilities could be used as initial part of a future demonstration project to show the feasibility of a system of several reservoirs. If the results of the components tests were completely satisfactory, it might be possible to go directly to a demonstration grant project to apply the system to the entire watershed. The treatment system and reservoir could also be transferred to the community if desired as a supplemental source of water. If further use of the system in Wilde Lake area is not desired, the reservoir and pretreatment system would be dismantled and the treatment plant transferred to another demonstration project. Alternately, the treatment plant would have a high market value and could be sold to recover the majority of its cost.

E. Demonstration of System Components - Alternate Plan

1. General

The "Demonstration of System Components" described in Section VII-D, would not include a demonstration of an interaction of a series-parallel arrangement of pollution control reservoirs and an evaluation of the effectiveness of the local storage reservoirs to control pollution for a large area. An alternate plan was considered in which the system features of pollution control of the "Demonstration of the Overall System" were combined with the package treatment demonstration concept of the previously discussed "Demonstration of System Components." The purpose of the alternate plan would be a program in which the pollution control aspects would be adequately demonstrated to permit applications of this part of the system, even though further demonstration work might be required before reuse could be generally applied.

2. System Description

The physical facilities for the "Demonstration of System Components - Alternate Plan" would be of the same design and at the same location as shown on Figure VII-1 and would consist of the following:

a. Gaging and Sampling Stations. This would be of the design shown in Appendix E. A gaging station would monitor flows and pollution concentration from the area to be controlled and the second would monitor an uncontrolled area.

b. Storage Reservoirs and Pretreatment Units. This would be located at Locations A, C, and D and would be of the same design as previously discussed.

c. Package Treatment System. This would be similar in design to the plant discussed in Section VII-D; however, this plant would have a capacity of 20 gpm. The plant would be sized to be capable of treating the water from the reservoir at Location A and after a satisfactory initial demonstration could be connected to the public system.

d. Pumping Facilities and Connection. The pumping facilities would be capable of supplying water to the public water system, and would not require the hydropneumatic tank and supply from the public water system, that was previously discussed for supplying a separate dual distribution.

Since the emphasis in the "Atlernate Plan" would be placed on pollution control, the dual distribution in the apartments would not be installed. Any actual demonstration of reuse would be through connection to the public system after first demonstrating the ability to produce reliably potable water.

3. Construction Plan

The "Demonstration of System Components - Alternate Plan" would have two increments of construction as shown in Figure VII-4. In the first increment, the gaging and sampling stations and the first reservoir would be constructed. The second increment would include the remaining two storage reservoirs and pretreatment system, the site facilities for the packaged treatment plants, the installation of the package treatment plant, and the connection to the public system.

4. Operation and Maintenance

The operation and maintenance program would involve 17 months operation of the gaging and sampling stations, 16 months operation of the first storage reservoir, 8 months operation of the three reservoirs, and 7 months of the treatment plant, and 4 months of supplying water to the public water system.

5. Test and Evaluation

The test and evaluation program would have the same tasks as previously discussed and the detailed item of work to be accomplished are listed in Appendix G.

6. Disposition of the Facilities

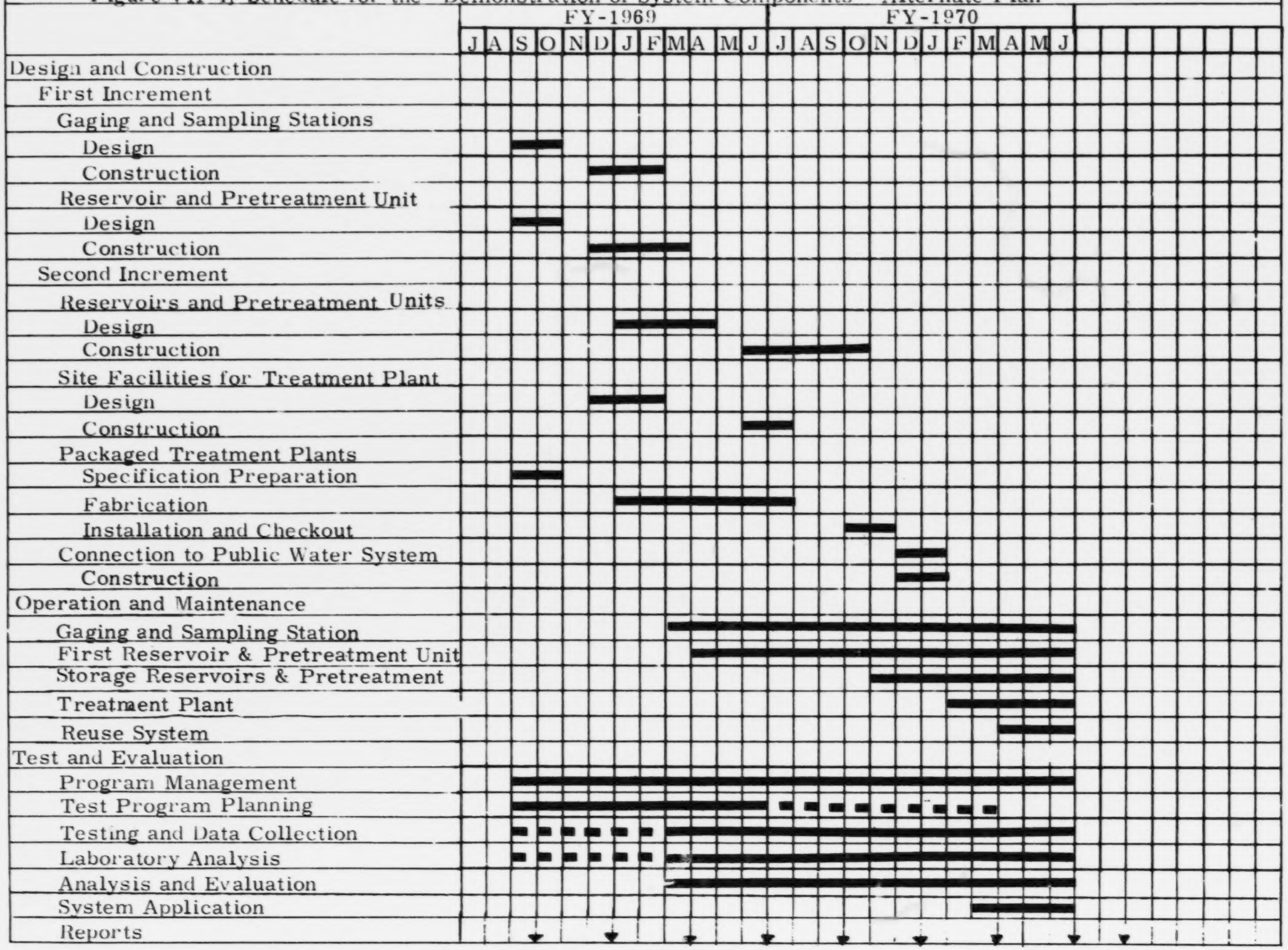
As previously discussed, the gaging and sampling stations would probably have long term value in the research programs on the effects or urbanization in the Columbia area. The three reservoirs could serve as the initial increment of a program to apply pollution control to the Wilde Lake watershed or as means of controlling erosion and pollution for a portion of the area. The treatment plant could be transferred to the community and county for further operation or have a substantial value for resale.

F. Combined Demonstration Program

1. General

This approach envisions a program in which the critical components of local storage, treatment, and reuse are first demonstrated on a limited scale

Figure VII-4. Schedule for the "Demonstration of System Components" Alternate Plan



and after satisfactory demonstration of the feasibility of the critical components, the system would be applied to the entire Wilde Lake watershed. The initial effort would be directed to the application of the local storage and treatment concept as an advanced method of pollution control. This would be followed with an effort to establish the feasibility of applying treatment facilities for the reuse of the storm water. This phase would concentrate on demonstrating the technical and economic feasibility of collecting and treating the storm water to drinking water standards for distribution through the existing potable water system.

Although this approach will require a longer period of time than the other two approaches, it will provide the greater base of technology for future applications of the local storage system for the solution of storm water and combined sewer pollution control problems. This approach has the advantage of providing tangible benefits to the community and in the form of a complete system suitable for long term use by the community.

2. System Description and Construction Plan

The "Combined Demonstration Program" will be a four phase program. The four phases are:

Phase 1. Demonstration of the Critical Components for Pollution Control

Phase 2. Application of Local Storage Pollution Control to the Wilde Lake Watershed

Phase 3. Demonstration of the Feasibility of Treating Storm Water to Drinking Water Standards

Phase 4. Application of Storm Water Treatment Plants to the Wilde Lake Watershed

The physical facilities required for this program of system demonstration and application would be a combination of the facilities previously described for the "Demonstration of the Overall System," Section VII-C, and the facilities for the overall system discussed in Chapter IV, Local Storage, Treatment, and Reuse System. The facilities requirements for each of the four phases listed above and the overall plan for construction of these facilities are described in the following sub-paragraphs. A location plan for these facilities is shown in Figure VII-5. An overall scheduling for the design, construction, test, and evaluation of the demonstration facilities is contained in Figure VII-6.

a. Phase 1. Demonstration of Critical System Components for Pollution Control. The establishment of the feasibility and determination of performance data for the critical components for effecting pollution control on a prototype unit basis are:

(1) Three storage reservoirs

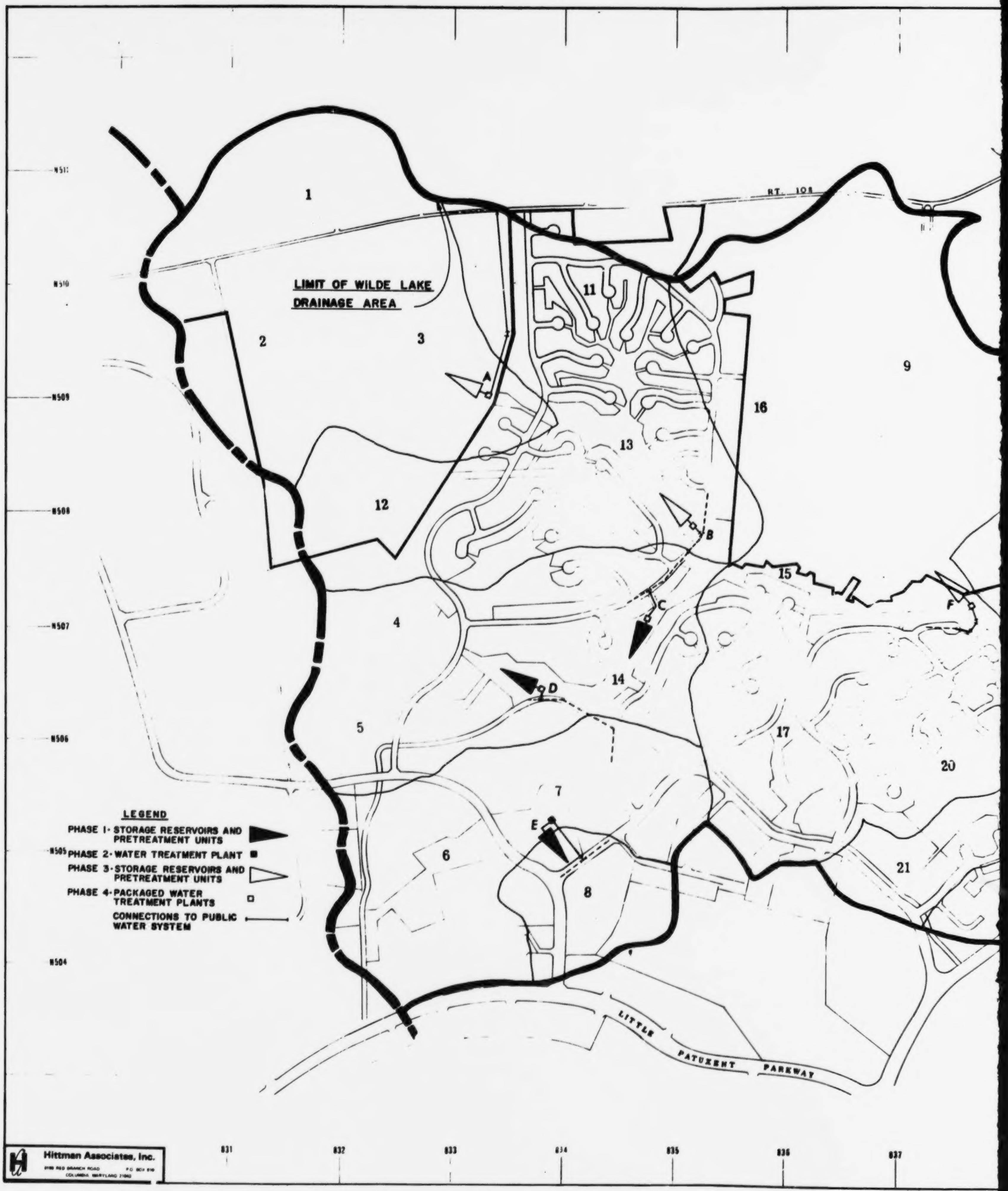
(2) Three pretreatment systems

These would consist of the three reservoirs shown in Figure VII-1 and discussed in detail in Section VII-C-2 and Appendix E. As in the case of the "Demonstration of the Overall System" approach, one reservoir would be constructed as a "lead unit" to obtain performance information prior to committing the two remaining reservoirs and pretreatment units to construction. The three reservoirs and pretreatment units would be used to demonstrate both the performance characteristics of the individual storage and treatment installations and the interrelationship of these installations when functioning as an integrated system.

The two gaging and sampling stations contemplated under the "Demonstration of the Overall System" approach would not be used. The individual installations would be equipped to record flows and sampling would be performed using portable equipment. The two gaging and sampling stations would not be installed since this approach envisions pollution control of the entire Wilde Lake watershed, and the evaluation of the effectiveness can be performed on the overall system. Likewise, any work on the treatment and reuse of storm water would be deferred until Phase 3 and after the pollution control aspects are thoroughly demonstrated.

b. Phase 2. Application of Local Storage Pollution Control to the Wilde Lake Watershed. After the operation of the three local storage reservoirs and pretreatment systems have satisfactorily demonstrated the effectiveness of the local storage and treatment system as a means of controlling storm water pollution, the planning and design work required to apply this concept to the entire watershed would be initiated. This phase would consist of the construction of seven reservoirs at the location shown on Figure VII-5. These seven reservoirs, plus the three constructed in Phase 1, will provide a complete pollution control system for Wilde Lake. These reservoirs would have the pretreatment units and would have the following characteristics:

Location	Reservoir Size	Pretreatment Capacity
A	1,577,300 gal.	67,900 gals/day
B	2,145,000 gal.	231,400 gals/day
F	5,101,400 gal.	1,036,300 gals/day
G	923,500 gal.	85,700 gals/day
H	358,400 gal.	29,800 gals/day
I	690,000 gal.	63,800 gals/day
J	375,000 gal.	41,300 gals/day


LIMIT OF WILDE LAKE DRAINAGE AREA
RT. 108
LITTLE PATUXENT PARKWAY
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21
A
B
C
D
E
F
N511
N510
N509
N508
N507
N506
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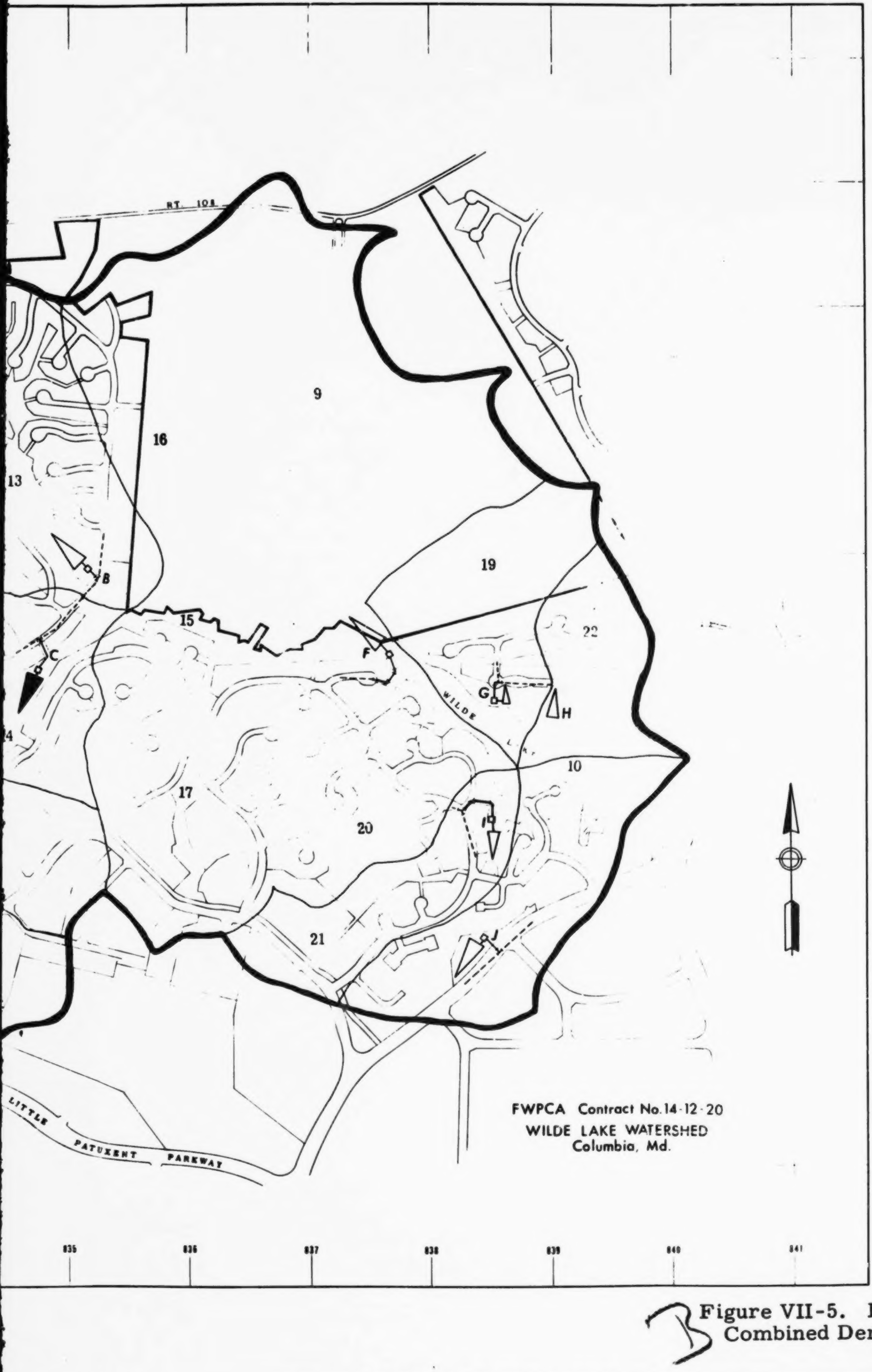
N504
831
832
833
834
835
836
837
LEGEND
PHASE 1- STORAGE RESERVOIRS AND PRETREATMENT UNITS
PHASE 2- WATER TREATMENT PLANT
PHASE 3- STORAGE RESERVOIRS AND PRETREATMENT UNITS
PHASE 4- PACKAGED WATER TREATMENT PLANTS
CONNECTIONS TO PUBLIC WATER SYSTEM
Hittman Associates, Inc.
RED BRANCH ROAD
COLUMBIA, MARYLAND

A


RT. 108
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PATUXENT
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841

FWPCA Contract No. 14-12-20
WILDE LAKE WATERSHED
Columbia, Md.

B

Figure VII-5. Location of Facilities - Combined Demonstration Program

Figure VII-6. Schedule for the "Combined Demonstration Program"

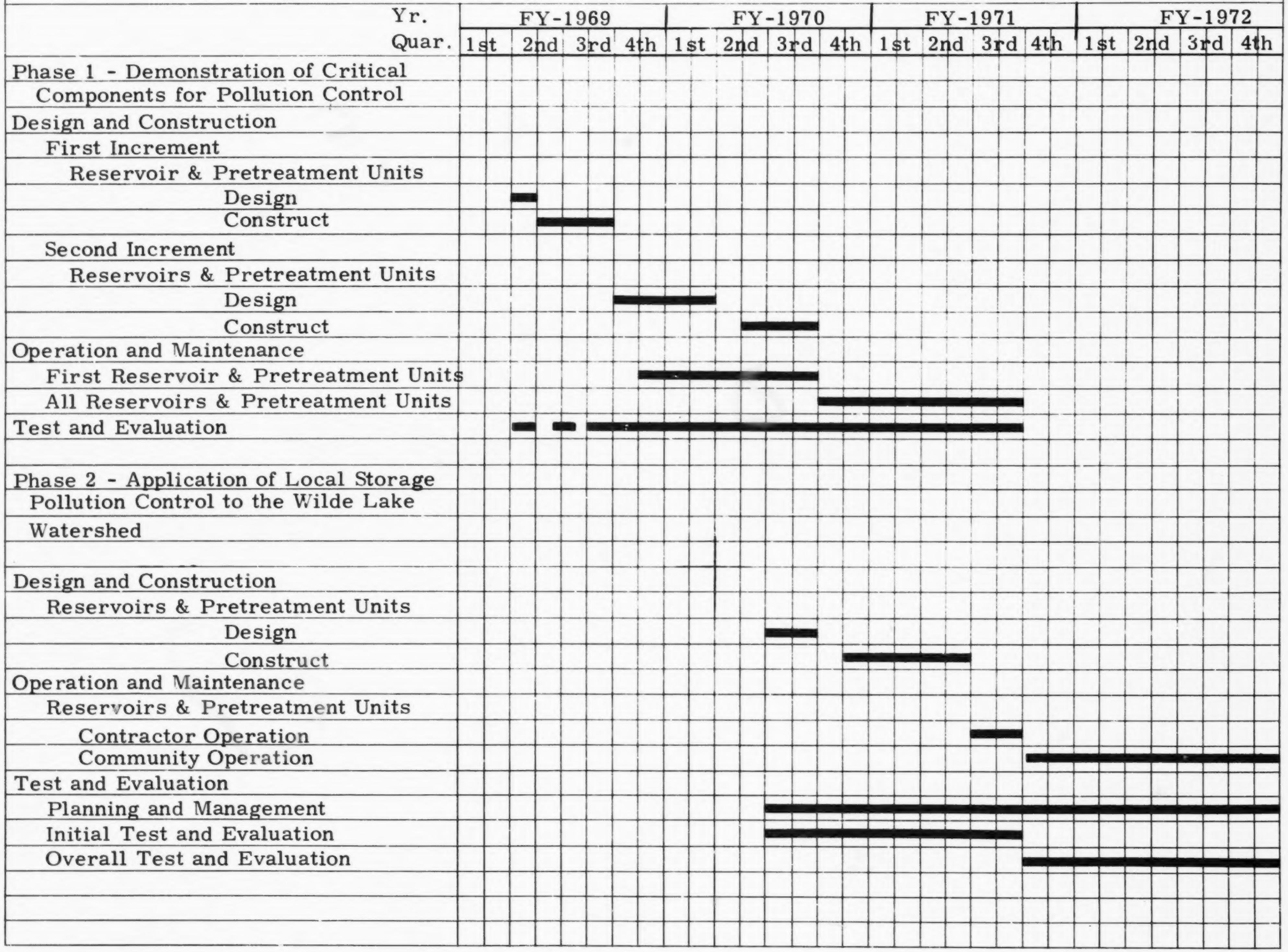
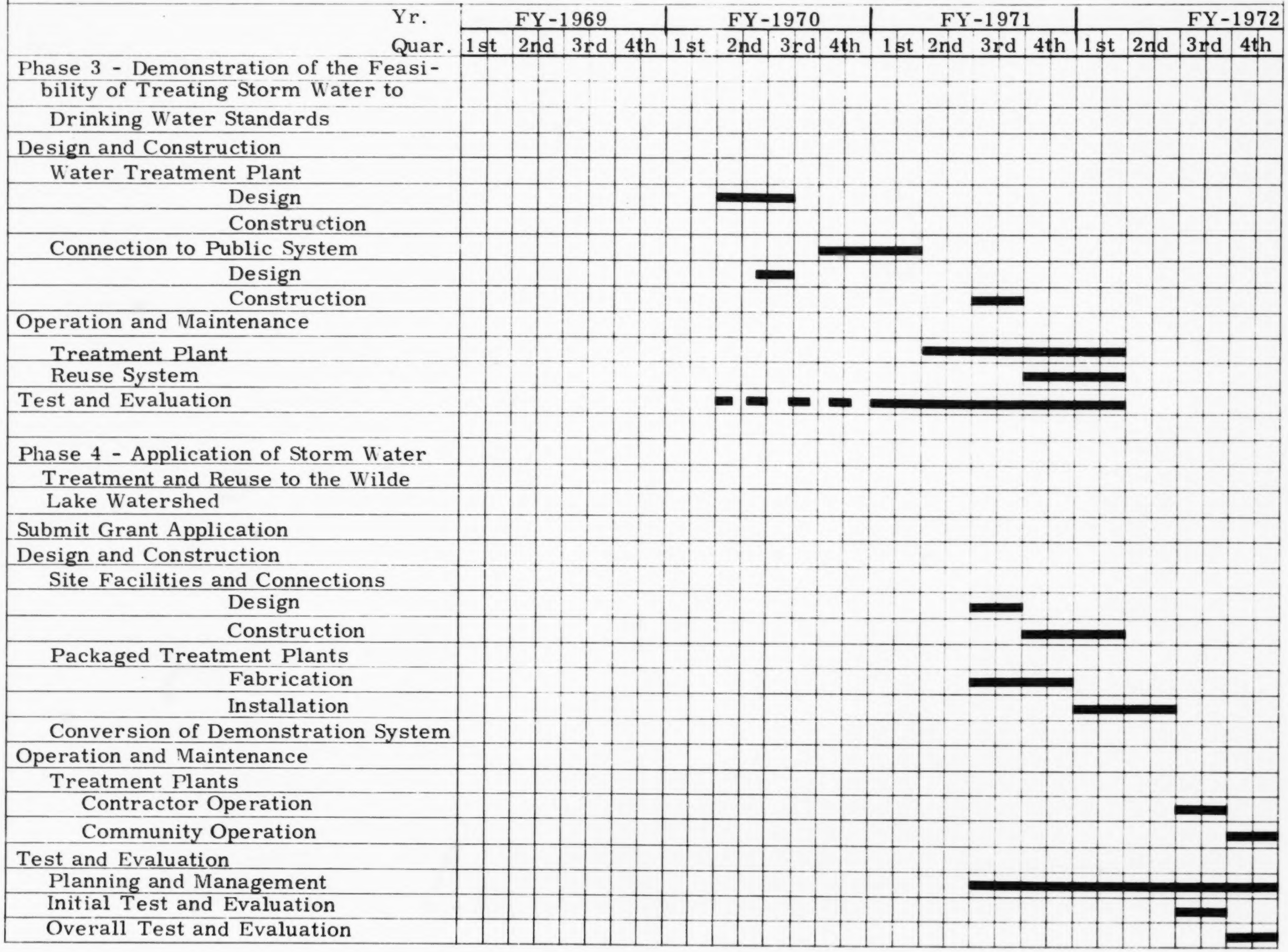


Figure VII-6. Schedule for the "Combined Demonstration Program" (Continued)



After these reservoirs are constructed and placed in operation, the entire system of reservoirs would be turned over to the community for operation and maintenance. A program for collection of information on quantity and quality of runoff and treated water would be continued to evaluate the system.

In the planning of these eight facilities, provision would be made for the future installation of treatment facilities under Phase 4 of this program.

c. Phase 3. Demonstration of the Feasibility of Treating Storm Water to Drinking Water Standards. In the later portion of the Phase 1, work on establishing the feasibility of the local storage and treatment system as a pollution control system, planning work would be initiated on the design of a demonstration treatment facility. The design of this facility would be the same as the design previously discussed under Section VII-C and D, and shown in Appendix E. This treatment facility would be a general test and evaluation facility in which various treatment processes could be evaluated. Although the "Combined Demonstration Program" approach would be primarily directed toward establishing the feasibility of treating storm water to drinking water standards, the test and evaluation program would include the technical and economic evaluation of treatment to lesser qualities in order to provide the information required to apply the local storage, treatment, and reuse system to other regions of the country.

The treatment facility would be constructed in parallel with the construction of the seven reservoirs under Phase 2 of this program. The test and evaluation program would be initiated upon the completion of the treatment facility. During the initial test and evaluation of the treatment facility, the treatment water would be discharged to the drainage stream and ultimately to Wilde Lake. As soon as the feasibility of reliably producing water of drinking water standards is demonstrated, and the necessary approvals of the state and local health departments are obtained, a connecting line to existing water distribution system would be constructed. The test and evaluation program will continue and will include collection of data and the effectiveness of the local storage, treatment, and reuse system as a supplemental source of potable water.

d. Phase 4. Applications of Storm Water Treatment Plants to the Wilde Lake Watershed. After the feasibility of reliably treating and distributing treated storm water is established, planning and design work would be started for the construction of additional treatment plants to be used to provide treated storm water to the majority of the Wilde Lake watershed. As now envisioned, this will consist of eight treatment plants. Not all of the local storage and treatment pollution control reservoirs will have treatment facilities since the available supply and the demands in certain areas does not justify the cost of such facilities. The present plan envisions the construction of the following treatment plants at the locations shown in Figure VII-5.

Location	Size of Plant
A	29,900 gals/day
B	81,800 gals/day
C	76,900 gals/day
D	86,800 gals/day
F	209,400 gals/day
G	23,100 gals/day
I	31,900 gals/day
J	20,700 gals/day

The designs for these treatment plants will be evolved during the Phase 3 portion of this program in which the various treatment processes will be evaluated. It is planned that these treatment plants will be designed to provide only the processes found best from this evaluation program and will be automated to the maximum extent to minimize operating personnel requirements. The treatment plant constructed under Phase 3 for evaluation will also be converted to a normal operating plant, and will be turned over to the community for operation and maintenance. The program of data collection will continue in order to obtain the information required for a complete system evaluation.

3. Operation and Maintenance

Under the "Combined Demonstration Program," the facilities would be operated under the test and evaluation portion of the demonstration projects until technical feasibility is demonstrated. The facilities would then be modified to make them suitable for long term use and would be transferred to the community for operation and maintenance. In the case of the pollution control reservoirs, there would be six months of operation on the first reservoir to obtain performance information for use in the design of the remaining two reservoirs. The three reservoirs will be operated for 18 months as a pollution control system after which they would be transferred to the community. As part of the demonstration grant program on pollution control, there would be an initial three month operating period to test and evaluate after which the system would be transferred to the community. Likewise, in the Phase 3 program for demonstration of the treatment of storm water, the treatment plant would be operated for six months to demonstrate the feasibility of producing potable water of drinking water standards. This would be followed by nine months of operation in which water was actually supplied to the public water system. The additional treatment plants constructed under the demonstration grant project would be operated for three months for test and evaluation and for three additional months supplying water to the public water system.

4. Test and Evaluation

As indicated above, the test and evaluation would parallel the operation of the facilities constructed under the two demonstration projects previously discussed. The same general task breakdown as previously discussed in Section VII-C-4 would apply to the demonstration contract program. The application projects would also have two test and evaluation tasks. These would be:

a. Planning, Management, and Coordination. This task would include the planning, technical direction, management, and coordination of the application projects.

b.. Initial Test and Evaluation. This task would cover the initial check-out, testing, and evaluation of the reservoirs and pretreatment system and the treatment plants constructed under the application projects.

c. Overall System Test and Evaluation. This task would cover the collection and analysis of data during the design and construction and after the overall storm water pollution control system and all of the treatment plants are placed into operation.

The items of work to be performed in each of the tasks under the "Combined Demonstration Program" are listed in Appendix G.

5. Disposition of the System

The "Combined System Demonstration" program is based on having all the facilities constrcuted under the demonstration and application projects become part of a permanent pollution control and supplementary water supply system for Wilde Lake.

G. Demonstration Program Costs

1. General

As discussed in Section VII-C, D, E, and F, each of the four approaches involved the design and construction of facilities similar in design. The principal differences in the four approaches is the number of facilities of each type constructed and the sequence in which the facilities are constructed. This in turn will affect the cost of operating and maintaining the facilities during the demonstration program and the scope of test and evaluation effort and the amount and type of information that will be obtained.

Three of the approaches are based on the demonstration program being conducted as a demonstration project. The fourth approach for the "Combined Demonstration Program" assumes a program in which the technical and economic feasibility of controlling pollution and water supply augmentation is first demonstrated as demonstration projects. Having demonstrated the feasibility of the local storage, treatment, and reuse system on a limited scale, it would then be applied to the overall watershed.

2. Overall Costs

For purposes of presenting costs, the demonstration program is considered to be divided into three major categories. These are further defined as follows:

a. Detailed Design and Construction. This includes the preparation of design drawings and specifications for obtaining lump sum bids for the construction of the facilities, preparation of cost estimates, analysis of bids, field engineering, and inspection services. This work will be performed by subcontractor with the exception of the detailed design of the internal reuse system which will be performed by the developer. The construction portion includes construction of the facilities; installation of the equipment, except for special instrumentation and includes an allowance for escalation and field changes. This work would be performed by subcontractors, except for the reuse system which would be performed by the builder of the apartments. This category would include provisions for modification to the system performed by subcontractors.

b. Operation and Maintenance. This covers the cost of labor, equipment rental, supplies and materials required to operate, perform preventive maintenance, and repair the facilities. This category also includes the maintenance and repair work that would be performed by subcontractors and under agreements with the community or county. This category excludes all labor, supplies, materials, and equipment used specifically for the test and evaluation of the system.

c. Test and Evaluation. This category includes the management and technical direction, planning and coordination of the overall demonstration program and the labor, materials, supplies, and equipment for the test and evaluation of the components and systems. The test and evaluation includes the planning and coordination of tests, conduct of tests and collection of data, supplying laboratory analysis, analysis and evaluation, system application and development work and the preparation of reports. It includes all procurement, installation, maintenance, and special instrumentation required for the test and evaluation program and the labor involved in the special tests.

Tables VII-1, VII-2, VII-3, and VII-4 contain summaries of the costs of the "Demonstration of the Overall System," the "Demonstration of System Components," the "Demonstration of System Components - Alternate Plan," and the "Combined System Demonstration."

A comparison of the costs for each of the approaches with a discussion of the technical merits of each is contained in Section VII-I, "Comparison of Demonstration Program Approaches," of this chapter.

TABLE VII-1. PROGRAM COSTS "DEMONSTRATION OF OVERALL SYSTEM"

1.	Design and Construction					
	a.	Detailed Design and Preparation of Specifications			$ 42,000	
	b.	Field Engineering and Inspection			26,500	
	c.	Construction				
		(1)	Gaging and Sampling Stations	$ 32,300		
		(2)	Storage Reservoir and Pretreatment Units	179,500		
		(3)	Treatment Plant	120,000		
		(4)	Connection to Apartments	11,000		
		(5)	Contingency and Escalation	68,500		
		SUBTOTAL			$411,300	
	d.	Design and Installation - Reuse Distribution System			21,000	
		SUBTOTAL				$500,800
2.	Operation and Maintenance					
	a.	Gaging and Sampling Stations			9,900	
	b.	Storage Reservoirs and Pretreatment Units			20,100	
	c.	Treatment Plant			18,800	
	d.	Reuse System			5,400	
		SUBTOTAL				54,200
3.	Test and Evaluation					
	a.	Program Management			78,700	
	b.	Test Program Planning			55,800	
	c.	Testing and Data Collection			81,800	
	d.	Laboratory Analysis			48,300	
	e.	Analysis and Evaluation			55,700	
	f.	System Application			27,700	
	g.	Reports			29,500	
		SUBTOTAL				377,500
4.	TOTAL					$932,500

TABLE VII-2. PROGRAM COSTS "DEMONSTRATION OF SYSTEM COMPONENTS"

1.	Design and Construction			
a.	Detailed Design and Preparation of Specifications		$ 12,000	
b.	Field Engineering and Inspection		7,800	
c.	Construction			
(1)	Storage Reservoir and Pretreatment Units	$ 70,400		
(2)	Connection to Apartments	11,000		
(3)	Site Facilities for Treatment Plant	7,500		
(4)	Pumping and Storage Facilities	10,200		
(5)	Contingency and Escalation for Reuse System	19,800		
	SUBTOTAL		$118,900	
d.	Design and Installation Reuse System		21,000	
e.	Fabrication, Installation, and Check-out of Packaged Treatment Plant		20,000	
	SUBTOTAL			$179,700
2.	Operation and Maintenance			
a.	Gaging and Sampling Stations		---	
b.	Storage Reservoir and Pretreatment Unit		11,200	
c.	Treatment Plant		16,700	
d.	Reuse System		5,400	
	SUBTOTAL			33,300
3.	Test and Evaluation			
a.	Program Management		54,800	
b.	Test Program Planning and Coordination		27,400	
c.	Testing and Data Collection		52,500	
d.	Laboratory Analysis		14,300	
e.	Analysis and Evaluation		32,100	
f.	System Application		14,400	
g.	Reports		13,400	
	SUBTOTAL			208,900
4.	TOTAL			$421,900

TABLE VII-3. PROGRAM COSTS
"DEMONSTRATION OF SYSTEM COMPONENTS - ALTERNATE PLAN"

1.	Design and Construction					
	a.		Detailed Design and Preparation of Specifications		$ 28,800	
	b.		Field Engineering and Inspection		17,500	
	c.		Construction			
		(1)	Gaging and Sampling Stations	$ 32,300		
		(2)	Storage Reservoirs and Pretreatment Units	179,500		
		(3)	Site Facilities for Treatment Plant	7,500		
		(4)	Pumping and Storage Facilities	10,200		
		(5)	Connection to Public System	2,000		
		(6)	Contingencies and Escalation	46,300		
			SUBTOTAL		277,800	
	d.		Fabrication, Installation and Check-out of Packaged Treatment Plant		20,000	
			SUBTOTAL			$342,300
2.	Operation and Maintenance					
	a.		Gaging and Sampling Stations		9,900	
	b.		Storage Reservoirs & Pretreatment Units		20,100	
	c.		Treatment Plants		14,000	
	d.		Reuse System		1,500	
			SUBTOTAL			45,500
3.	Test and Evaluation					
	a.		Program Management		70,700	
	b.		Test Program Planning and Coordination		55,800	
	c.		Testing and Data Collection		80,100	
	d.		Laboratory Analysis		48,300	
	e.		Analysis and Evaluation		52,500	
	f.		System Application		27,700	
	g.		Reports		29,500	
			SUBTOTAL			364,600
4.	TOTAL					$752,400

TABLE VII-4. PROGRAM COSTS "COMBINED DEMONSTRATION PROGRAM"

PHASE 1 - DEMONSTRATION OF CRITICAL COMPONENTS FOR POLLUTION CONTROL

1.	Design and Construction				
	a.		Detailed Design, Specification Preparation, Field Engineering and Inspection	$ 31,000	
	b.		Construction		
		(1)	Storage Reservoirs and Pretreatment Units	153,500	
		(2)	Contingencies and Escalation	31,000	
			SUBTOTAL		$215,500
2.	Operation and Maintenance				23,100
3.	Test and Evaluation				287,900
4.	TOTAL				$526,500

PHASE 2 - APPLICATION OF LOCAL STORAGE POLLUTION CONTROL TO THE WILDE LAKE WATERSHED

1.	Design and Construction				
	a.		Detailed Design, Specification Preparation, Field Engineering, and Inspection	$ 92,400	
	b.		Construction		
		(1)	Storage Reservoirs and Pretreatment Units	615,000	
		(2)	Contingencies and Escalation	61,500	
			SUBTOTAL		$768,900
2.	Operation and Maintenance				29,600
3.	Test and Evaluation				200,700
4.	TOTAL				$999,200

TABLE VII-4. PROGRAM COSTS "COMBINED DEMONSTRATION PROGRAM" (Continued)

PHASE 3 - DEMONSTRATION OF THE FEASIBILITY OF TREATING STORM WATER TO DRINKING WATER STANDARDS

1.	Design and Construction				
	a.	Detail Design, Specification Preparation, Field Engineering, and Inspection		$ 24,000	
	b.	Construction			
		(1)	Treatment Plant	120,000	
		(2)	Connection to Public System	2,000	
		(3)	Contingencies and Escalation	24,000	
		SUBTOTAL			$170,000
2.	Operation and Maintenance				21,800
3.	Test and Evaluation				255,400
4.	TOTAL				$447,200

PHASE 4 - APPLICATION OF STORM WATER TREATMENT PLANTS TO THE WILDE LAKE WATERSHED

1.	Design and Construction				
	a.	Detailed Design, Specification Preparation, Field Engineering, and Inspection		$ 57,500	
	b.	Construction			
		(1)	Water Treatment Plants	449,700	
		(2)	Contingencies and Escalation	45,000	
		SUBTOTAL			$552,200
2.	Operation and Maintenance				46,700
3.	Test and Evaluation				152,600
4.	TOTAL				$751,500

H. Coordination with Local and State Organizations

Any demonstration program of pollution control and the local storage and treatment concept for storm water reuse in the Wilde Lake watershed must involve the participation of The Rouse Company, as the developer of Columbia; the Columbia Park and Recreation Association; the Howard County Metropolitan Commission; the Howard County Planning Commission; the Howard County Commissioners; the State of Maryland, Department of Health; and the State of Maryland, Department of Water Resources. Depending upon the approach finally selected for the demonstration, the participation of these groups can involve either concurrance and/or approval of the construction of facilities under a demonstration contract, direct involvement in the planning, construction, operation, maintenance, and evaluation of the demonstration program or a financial contribution to the program.

During the course of this program, contact has been maintained with the various organizations that would be involved in a demonstration program. Several presentations of the interim results of the study have been made and the final report will be made available for review. At that time, formal statements of interest and agreements to participate in the program will be obtained. The comments received have been primarily oral and are summarized in the following paragraphs.

1. The Rouse Company

The Rouse Company through the Howard Research and Development Corporation is responsible for the overall planning and development of Columbia, Maryland. The Rouse Company has been involved in the study of the local storage, treatment, and reuse of storm water from the initiation of the program. The Rouse Company has previously agreed to a demonstration project in Columbia subject to the final approval on the plan for construction of the necessary facilities. The Rouse Company has provided the maps, drawings, and development plans of the Columbia area that were used in the system study and design and also provide a site for the temporary gaging station used to collect data during the study.

The Rouse Company is currently working with the Howard County Soil Conservation Office on a program to measure the sediment accumulation in Wilde Lake. They are also cooperating with the University of Maryland on a research program to determine the effects of urbanization on the micrometeorology of the area. The Rouse Company is interested in having the local storm water pollution control demonstration project conducted in the area since it would complement other research work.

The results of the local storage, treatment, and reuse of storm water have been reviewed with various representatives of The Rouse Company. They are generally interested in the use of the system not only in the Wilde Lake area but in the future villages of Wilde Lake. Wilde Lake has been dredged on two occasions and there have been some problems with erosion in Wilde Lake Village. They are particularily interested in the possibility of installing such a system prior to the development of an area and controlling the sedimentation during the construction and post-construction periods. They are also interested in the use of storm water as a supplementary source of water.

With respect to the design and construction of the demonstration facilities, The Rouse Company will want to review the plans to assure that the aesthetic standards of the community are maintained. The Rouse Company has made the building plans for the apartment buildings at Harpers Farm-East available for use in the study. They would be willing to allow a dual water distribution system to be installed in these buildings if a demonstration of actual reuse is desired and provided that Public Health Department has no objections.

The Rouse Company believes that a demonstration of the local storage system will be required to establish the economic and technical feasibility. The possibility of conducting such a program under a demonstration grant with direct or financial participation of The Rouse Company has been explored and a formal commitment on the extent of such participation is being obtained.

2. Columbia Park and Recreation Association

The Columbia Park and Recreation Association, which is also referred to as the Columbia Association, is a nonprofit corporation established and empowered to perform pseudo-governmental functions during and after the development of Columbia. This corporation has title to all of the dedicated open land spaces of Columbia and is responsible for the maintenance of all of the parks, recreation areas, roads, lakes, sidewalks, and the storm water collection system in Columbia. The Columbia Association receives revenue from the residents and business establishments in the Columbia area through covenants on the leases and deeds for property. As Columbia becomes more fully developed it is envisioned that the Columbia Association Corporation could be replaced by some form of local government and the covenants on leases and deeds may be replaced by some form of local tax structure.

The Columbia Association is now participating in various demonstration projects with government agencies and universities. The largest such project is a grant program, the first phase of which is to evaluate new concepts for transportation system and to select the one best suited to Columbia. In a second phase, an experimental minibus system will be put into operation. This $416, 600 project is being performed on a grant of $277, 733 from the Department of Housing and Urban Development.

The results of the system study on the "Local Storage, Treatment, and Reuse of Storm Water" were reviewed with the Columbia Association along with the possible approaches for demonstrating this system in the Wilde Lake watershed.

(a) The Association is most concerned with the abatement of erosion within the Wilde Lake watershed and the improvement of the quality of water in Wilde Lake.

(b) The Association believes the concept has a great potential not only to Wilde Lake but for the future lakes and villages of Columbia.

(c) The prospect of reusing storm water is of interest since the Columbia region does not have any substantial ground water supplies and future water requirements will require import of water or development of surface water supplies.

(d) Since neither the pollution control aspects nor the reuse aspects have been demonstrated, it is considered that such a demonstration should proceed any general application of the concept.

(e) The Association would be willing to cooperate with the demonstration of the concept as proposed for the Wilde Lake area, but would want to have final approval on the location and aesthetic aspects of the reservoirs and treatment facilities.

Since the Columbia Association is not truly a governmental body, it is questionable whether they could be the recipient of the grant for a demonstration project. Possible financial participation or direct performance of certain items of work on a demonstration project by the Columbia Association is being discussed.

3. The Howard County Metropolitan Commission

The Howard County Metropolitan Commission is responsible for the planning, design, construction, operation, and maintenance of the water supply and sanitary waste disposal system of the county. For the portion of the county in which Columbia is located, the County purchases water from the City of Baltimore and the water is distributed through a main distribution system provided by the County to the major developments with the County. Local distribution systems within the developments are constructed by the developers in accordance with the requirements of the County. After satisfactory completion and inspection, these systems are accepted by the County for future operation and maintenance. The Metropolitan Commission owns and operates a wastewater treatment plant at Savage, Maryland, to which Columbia is connected by sewers owned and operated by the County. The Metropolitan Commission sources of revenue are a charge of 45 cents per thousand gallons for water and a charge of 40.5 cents per thousand gallons for sanitary waste collection and treatment.

The authority of the Howard County Metropolitan Commission is confined to water supply and distribution and wastewater collection and treatment. The Commission is not involved in storm water drainage nor erosion control. As such, their interest would be confined to the use of storm water as a supplementary water supply for Howard County. The Metropolitan Water Commission is most interested in the development of water resources within the County and have requested additional information on the cost of treating storm water. They have agreed to consider possible participation in the reuse aspects of a demonstration program. The distribution of treated storm water through the public water distribution system has been discussed. There would probably be no objection if the ability to produce high quality water was satisfactorily demonstrated and there were no objections from the Department of Health.

4. Howard County Soil Conservation Service

The organization primarily involved with storm water drainage, flood control, and sedimentation control is the Soil Conservation Service of the U. S. Department of Agriculture. This group prepares standards on the storm drainage and sedimentation control and reviews plans for land development and reservoir construction. Within Howard County, the Soil Conservation Board is responsible for the approval of policy standards, enforcement of regulations, and the initiation of projects for consideration by the County Commissioners. The Soil Conservation Service is most interested in the control of erosion and sedimentation during the development of Columbia and would be most interested in having the local storage and treatment system demonstrated in Columbia. Presentation to the Soil Conservation Board and the state representatives of the Soil Conservation Service are planned to solicit possible support and participation in the demonstration program.

5. Department of Health, State of Maryland

The State of Maryland, Department of Health, has the overall responsibility within the state for assuring the safety and quality of drinking water. The local storage, treatment, and reuse of storm water was discussed with the Department of Health and information regarding the proposed demonstration program was formally submitted for review and comment. The body of their letter reply is quoted as follows:

> "We have reviewed the information contained in your letter of July 18, concerning your plans for conducting a demonstration project on the 'local storage, treatment and reuse of storm water.' We agree that this proposed project has merit and may provide valuable information, not only for the control of storm water and sedimentation, but could evidently provide a valuable source of potable water supply.
>
> We note that you plan to provide adequate treatment so that a potable supply will be made available even though independent lines will be constructed to three apartment buildings where the water will be used for toilet flushing only. As long as the distribution of this water during the demonstration period is limited to the three apartment buildings, and as long as they continue to be owned and operated by the Rouse Company where strict controls can be exercised, we see no objection to your plan.
>
> As your proposal is of great interest to the State Department of Health, you can count on our support for the demonstration project and our assistance if we can help you in any way in securing Federal approval.
>
> Very truly yours,
>
> Thomas W. Shives, Chief
> Division of Water and Sewerage"

It is noted that in the discussions of the local storage, treatment, and reuse system with the Department of Health, it was indicated that the treatment of storm water to potable quality for distribution through the public water system was preferred to sub-potable water distributed through a separate distribution system.

6. Department of Water Resources, State of Maryland

The Department of Water Resources of the State of Maryland has the overall responsibility for maintaining the quality of the streams, lakes, and other bodies of water within Maryland. This department is responsible for the establishment of water quality standards and the enforcement of compliance with the standards. The Department of Water Resources is the group primarily concerned with industrial waste treatment and disposal, agricultural wastes, erosion and sedimentation, and effect of effluents from sanitary waste treatment plant on quality of lakes and streams. As such, the Department of Water Resources would be the cognizant State organization on the pollution control aspects and the use of storm water to augment water supplies. The health aspects of reuse would be the responsibility of the Department of Health. Accordingly, the Department of Water Resources would have the broadest interest in having the "Local Storage, Treatment, and Reuse System" demonstrated for future applications within the State and the County.

In discussions with representatives of the Department of Water Resources, considerable interest has been shown in the local storage and treatment concept. The State of Maryland has funds for participating in demonstration projects with the Local and Federal Governments. In the absence of a governmental group within Howard County to conduct such a demonstration program, the Department of Water Resources would be willing to consider making the grant application and directing and coordinating the demonstration program providing local participation can be arranged.

I. Comparison of Demonstration Program Approaches

The ultimate goal of the demonstration program for the "Local Storage, Treatment, and Reuse of Storm Water" will be the development of the necessary information to permit this system to be generally applied for the solution of storm water and combined sewer problems. The selection of the approach to be followed for the demonstration program must consider the extent to which the program will provide this information or whether the initial demonstration program must be followed by further demonstration programs to attain the ultimate goal. In addition, the time and overall cost of attaining the ultimate goal are important considerations.

Table VII-5, "Attainment of Program Objectives," is a listing showing the relative degree to which each of the approaches attains the primary and secondary objectives established for the demonstration program. Table VII-6, "Demonstration Program Costs," contains summary comparison of the costs for each of the programs approaches. The schedules for each approach are contained in Figures VII-2, 3, 4, and 6 which were previously discussed. The primary factors to be considered are as follows.

1. Combined Demonstration Program

This approach provides the best overall attainment of the objectives established for the program. By initially demonstrating the critical components of the system and then applying the system to an entire watershed, all aspects of the system will have been demonstrated adequately to permit other communities to apply the local storage system without the need for further demonstrations. The disadvantage of this approach is that the program will require four years to complete and will have an overall cost in excess of 2.7 million dollars. The cost of this program would be reduced by any contributions from the state or local government and would yield a permanent system for future use and for obtaining long term operating data.

2. Demonstration of Overall System

This approach is the second best attainment of the program objectives and will provide a better evaluation of the effectiveness of the system than the "Combined Demonstration Program" in that the gaging and sampling stations will provide for a comparison of controlled and uncontrolled areas. This approach should also provide the necessary information to permit the system to be applied by other communities without the need for further demonstration. Compared to the "Combined Demonstration Program," this demonstration program has the advantage that it can be completed within two years and at a much lower overall cost. The disadvantage is that the overall cost of the program is still quite high.

3. Demonstration of System Components - Alternate Plan

This approach provides the next best attainment of the program objectives

TABLE VII-5. ATTAINMENT OF PROGRAM OBJECTIVES

	Primary Objectives	A	B	C	D
a.	Demonstrate advanced treatment methods for treating storm water	**	*	**	***
b.	Demonstrate capabilities of series of local storage reservoirs to control pollution	**	-	**	***
c.	Demonstrate ability to dampen storm water flows and abate erosion	**	*	**	***
d.	Demonstrate acceptability of introducing local reservoirs from aesthetic standpoint	**	*	**	***
e.	Demonstrate ability to treat storm water to qualities acceptable for domestic usage	**	*	*	***
f.	Demonstrate acceptability of storm water reuse by the public and local authorities	**	**	*	***
g.	Demonstrate the capability of using storm water to supply substantial portion of water needs	**	**	*	***
h.	Define construction, operating, and maintenance costs and economic benefits	**	*	*	***
i.	Demonstrate effectiveness and importance of using local storage system to control pollution	**	*	**	***
	Secondary Objectives				
a.	Verification of hydrograph synthesis procedure for ungaged watersheds	***	*	***	***
b.	Verification of procedure to determine increase in runoff due to urbanization	***	*	***	**
c.	Verification of analysis methods used to calculate peak flow	***	*	***	**
d.	Collection of data on erosion and sedimentation due to urbanization	***	*	***	***
e.	Collection of data on the type and amount of pollutants during urban development	***	*	***	**
f.	Collection of data on pollutant content as a function of storm intensity and duration	***	*	***	**
g.	Determination of the performance characteristics of inclued tube settler with storm water	***	**	***	***
h.	Evaluation of the effectiveness of chemical addition with tube settlers	***	**	***	***
i.	Comparison of the impact of local storage system pollution control with uncontrolled areas	***	*	***	**
j.	Verification of the methods used in the design and szing of reservoirs	**	*	**	***
k.	Verification of the methods used to size interrelated reservoirs	**	-	**	***
l.	Determination of the performance characteristics of various treatment systems	**	*	*	***
m.	Determination of the problem involved in the operation of resue systems	***	**	*	***

TABLE VII-5. (Continued)

		A	B	C	D
n.	Collection of data on the internal usage of domestic water	**	**	*	***
o.	Establishment of cost of constructing, operating, and maintaining local storage system	**	*	**	***
p.	Evaluation of the local storage system for control of nutrient pollution	**	*	**	***

A Demonstration of Overall System
B Demonstration of System Components
B Demonstration of System Components - Alternate Approach
C Combined Demonstration Program

*** Full attainment of objective
** Moderate attainment of objective
* Partial attainment of objective
- Virtually no direct attainment of objective

TABLE VII-6. DEMONSTRATION PROGRAM COST SUMMARY
(Amounts in Thousand Dollars)

Demonstration Projects	Demonstration of the Overall System	Demonstration of System Components		Combined Demonstration Program	
		"Reference Plan"	"Alternate Plan"	Pollution Control	Storm Water Reuse
Design and Construction	500.8	179.7	342.3	215.5	170.0
Operation and Maintenance	54.2	33.3	45.5	23.1	21.8
Test and Evaluation	377.5	208.9	364.6	287.9	255.4
TOTAL	932.5	421.9	752.4	526.5	447.2
Application Projects					
Design and Construction				768.9	552.2
Operation and Maintenance				29.6	46.7
Test and Evaluation				200.7	152.6
TOTAL				999.2	751.5

and is nearly comparable to the "Demonstration of Overall System," in this respect. Compared to the "Demonstration of the Overall System," the use of packaged treatment plant will not permit a thorough evaluation of various treatment processes. The actual demonstration of reuse would depend upon first demonstrating the ability to make potable water of drinking water standards using a packaged treatment plant. By including three reservoirs and pretreatment systems and the two gaging and sampling stations, the system aspects of pollution control will be thoroughly demonstrated and the local storage system could be applied as a pollution control system by other communities without further demonstration. The cost of this approach is substantially reduced compared to the "Demonstration of the Overall System" and the program can be completed within two years. The cost could be further reduced due to the resale value of the package treatment plant and other equipment.

4. Demonstration of System Components

This approach provides for the lowest level of attainment of the overall program objectives and represents the minimum program that could be undertaken to demonstrate the critical components of the system. A future demonstration program would be required before the local storage system could be applied to other areas. However, in this approach, the technical feasibility of the various components and the acceptability of reuse are demonstrated. The prime advantage of this approach is that it requires the least commitment of funds to establish basic feasibility of the local storage system. The cost could be further reduced by the resale value of the packaged treatment plant and other equipment.

J. Conclusions and Recommendations

It is considered that this system study has shown that the local storage, treatment, and reuse of storm water is a practical and economically attractive method of providing pollution control in urban areas. Although the design and analysis of the local storage system used the best available methods or the best methods that could be developed during the course of the program, various important assumptions were necessary. These included such items as: the amount of runoff as a function of urbanization, the pollutant content of the runoff, the performance characteristics of inclined tube settlers when applied to storm water runoff, the sizing of storage reservoirs, the ability to adequately treat storm water runoff, the interactions of system components, and the economic aspects involved in the construction and operation of a system dispersed throughout an entire watershed. From a purely technical standpoint, it must be concluded that the present design and analysis work must be supplemented by an actual demonstration before this concept can be generally applied to the solution of storm water and combined sewer pollution problems.

Of the four approaches presented, the "Demonstration of System Components - Alternate Plan" is considered to represent the best overall approach for the demonstration program. This approach provides for the demonstration of the system aspects of pollution control and by the use of gaging and sampling stations the effectiveness and impact of the local storage and treatment system can be thoroughly and quantitatively evaluated. This approach is considered to yield the maximum amount of information and the best attainment of program objectives with respect to the time and cost of the program. This program combined with the other research programs in the Columbia area on the effects of urbanization should provide a major contribution to the general field of urban technology. This would be in addition to prime purpose of demonstrating the local storage system sufficiently to permit systems of this type to be generally applied.

All of the demonstration programs presented were based on demonstrating the local storage and treatment concept at an early date to permit application of this concept in the near future. However, as a result of the recent discussions with local and state organizations with respect to participation in the demonstration program, it has been concluded that a greater participation of the local and state organizations in the initial demonstration program could facilitate future application of the local storage concept even though the initial demonstration program might take longer. Accordingly, it is recommended that further consideration be given to phased demonstration programs in which the local and state organizations would participate to a greater extent. Using the "Demonstration of System Components - Alternate Plan" as the basic overall approach, the increments of such a program would be as follows:

1. One Local Storage Reservoir and Pretreatment Unit

This would permit the performance characteristics of the tube settler to be evaluated prior to committing to the installation of subsequent installations. In addition, the runoff from a single sub-watershed would be gaged and the pollutant content measured before additional reservoirs were designed and constructed.

2. Packaged Water Treatment Plant

A single package would be used to establish the feasibility of treating storm water from the pretreatment unit to potable water quality.

3. Additional Storage Reservoirs and Pretreatment Units

Two additional storage reservoirs and pretreatment units would be constructed after the satisfactory demonstration of the first installation. These reservoirs would permit the demonstration of the system aspects of a series-parallel arrangement of local storage and treatment installations and would provide the necessary information to permit the local storage and treatment concept to be applied as a method of pollution control.

4. Gaging Stations

With three reservoirs, a sufficiently large area would be controlled to permit the effectiveness of an overall system to be evaluated. A gaging station could then be installed in an adjacent uncontrolled watershed of comparable area to obtain comparative data. A gaging station for the controlled area could be eliminated by incorporating level recorders in the downstream reservoir and the pretreatment overflow weir.

5. Connection to Public Water System

After satisfactory demonstration of the packaged water treatment plant, a connection to the public water system could be installed for a demonstration of the actual reuse of storm water.

It is planned to use such a phased program in the future discussions with the local and state organizations. A definitive program will be recommended based upon the extent to which the local and state organizations are willing to participate in the demonstration program.

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APPENDIX A

DERIVATION OF HYDROLOGY EQUATIONS

Section II. C of this report, "Storm Water Hydrology" contains a discussion of the methods used to describe the storm flow characteristics of the Wilde Lake drainage area. This appendix contains the derivation of the equations used to modify the regression model for calculating the runoff for individual storms which are contained in Section II. C .4. These derivations are as follows.

1. Alteration of the Regression Equation

Let Q_{IP} = runoff/unit area from Herring Run

Q_P = runoff/unit area from the pervious portion of Herring Run or Wilde Lake

Q_I = runoff/unit area from the impervious portion of Herring Run

$$Q_{IP} = BR_i + CR_i^2 + DR_i^3 \qquad \text{(neglecting base flow)}$$

$$Q_{IP} = Q_P(1-I) + Q_I I$$

$$Q_I = KXR_i$$

$$Q_P(1-I) + Q_I I = BR_i + CR_i^2 + DR_i^3$$

$$Q_P = \frac{1}{1-I}(BR_i + CR_i^2 + DR_i^3) - \frac{KXR_i I}{1-I}$$

$$Q_P = \frac{1}{1-I}\left[(B-KXI)R_i + CR_i^2 + DR_i^3\right]$$

Let $Q_{I'}$ = runoff/unit area from the impervious portion of Wilde Lake

$Q_{I'P}$ = runoff/unit area from Wilde Lake

$$Q_{I'} = KXR_i$$

$$Q_{I'P} = Q_P(1-I') + Q_{I'}I'$$

$$Q_{I'P} = KXR_i I' + \left(\frac{1-I'}{1-I}\right)\left[(B-KXI)R_i + CR_i^2 + DR_i^3\right]$$

Adding the base flow term:

$$Q_i = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right)\left[(B-KXI)R_i + CR_i^2 + DR_i^3\right]$$

2. Derivation of the Expression Defining X

$$Q_I = \frac{(I+1)^{0.339}(R)^{1.75}(API)^{0.0741}(S)^{0.485}}{D^{0.0973}}{}^{*}$$

where:

I = percent imperviousness

R = precipitation in inches

API = antecedent precipitation index

S = slope of drainage channel

D = duration of storm

Q_I = runoff volume in inches from an area with I percent imperviousness

Let Q_o = runoff for I=0, and let X=% of rainfall on impervious areas that actually runs off. Then:

$$Q_I = \frac{XRI}{100} + Q_o\left(\frac{100-I}{100}\right)$$

$$Q_I - Q_o = \frac{XRI}{100} - \frac{Q_o I}{100}$$

$$X = \frac{100}{RI} Q_I \left[1 - \frac{Q_o}{Q_I}\left(1 - \frac{I}{100}\right)\right]$$

$$\frac{Q_o}{Q_I} = \frac{1}{(1+I)^{0.339}}$$

$$X = \frac{Q_I}{R}\left[\frac{100}{I} + \left(\frac{I-100}{I(1+I)^{0.339}}\right)\right]$$

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3. Derivation of the Equation Defining P

Let P = percent increase in unit yield of the developed area over the natural area.

$$P = \frac{Q_{I'P} - Q_P}{Q_P}$$

where:

$$Q_{I'P} = A + KXI'R_i + \left(\frac{1-I'}{1-I}\right) \quad (B-KXI)R_i + CR_i^2 + DR_i^3$$

$$Q_P = A + \left(\frac{1}{1-I}\right) \quad (B-KXI)R_i + CR_i^2 + DR_i^3$$

$$\text{Let} \left[(B-KXI)R_i + CR_i^2 + DR_i^3\right] = Y$$

$$P = \frac{A + KXI'R_i + \left(\frac{1-I'}{1-I}\right) Y - A + \left(\frac{1}{1-I}\right) Y}{A + \left(\frac{1}{1-I}\right)Y}$$

$$P = \frac{I'\left[KXR_i - \left(\frac{1}{1-I}\right)Y\right]}{A + \left(\frac{1}{1-I}\right)Y}$$

Simplifying:

$$P = \frac{I'\left[KX - B - CR_i - DR_i^2\right]}{\frac{A(1-I)}{R_i} - KXI+B+CR_i+DR_i^2}$$

APPENDIX B

WATER USAGE REPORT

Section II.D, "Water Quality and Demands," contains a discussion of the literature search made on the use of sub-potable water systems and dual distribution systems, water use classification, and the distribution of water demands throughout the Wilde Lake watershed. As part of this effort, a parallel study was made by Whitman, Requardt and Associates of the potential water demands as a function of water quality. This study is summarized in six tables, which are here reproduced from their report.

TABLE B-1. POTENTIAL WATER REUSES - QUALITY REQUIRED

Potential Reuses*	Quality of Water Required (By Water Use Category) "A-A"	"A"	"B"	"C"	Use Classification Consumptive	Nonconsumptive
Street Cleaning	Note: Water of Either class "A-A" or "A" should not be considered for potential reuses for the first nine inclusive because the cost of treatment would not be proportioned to the quality required.		o	-		x
Sewer Flushing			o	-	x	
Toilet Flushing			o	-	x	
Commercial and Industrial			o	-	x	x
Air conditioning			o	-	x	x
Fire Protection			o	-	x	x
Snow Melting			o	-		x
Ground Water Recharging			o	-	x	
Irrigation of Parks and Public Golf Courses			o	-	x	
Lawn Sprinkling			o	-	x	
Car Washing			o	-	x	x
Public Swimming Pools	o	-			x	
Private Swimming Pools	o	-			x	
Food Preparation	o				x	
Laundering and General Cleaning	o				x	
Human Consumption	o				x	
Cost of Treatment per 1000 gallons	.35	.30	.22	.19		

o - Quality Desired
- - Minimum quality allowed
* - Potential Reuses are listed in order of increasing quality requirements

TABLE B-2
POTENTIAL WATER REUSES - QUANTITY REQUIRED

Potential Reuse	Quantity of Water Required
Street Cleaning	200 gallons per city block, twice monthly
Sewer Flushing	250 gallons/100 l.f. sewer, 12-inch diameter and less once per year
Toilet Flushing	24 gallons per capita per day
Industrial and Commercial (Process Water)	Data from specific process must be considered
Air Conditioning	Without recirculation - 88 gallons/hour/ton With recirculation - 2% of demand without recirculation Consider recirculation at 5 tons capacity and above (See Table B-5)
Fire Protection	
Residential Area	500 gpm for duration of 4 hours
Commercial Area	2500 gpm for duration of 10 hours
Industrial Area	
General Warehouse	Sprinkler - 2500 gpm (1500 - sprinkler system int.)(1000 - hydrants exterior)
Special	As required
Snow Washing of Streets	As required - estimated 500 gpd for 30 minutes per city block
Ground Water Recharging	Estimated reasonable recharge - domestic demand (for areas served by wells)
Irrigation of Parks and Public Golf Courses	1 inch of water per week (during dry period) Less effective rainfall .33 inches of water per week (during 100-day growing season)
Private Lawn Sprinkling	1 inch of water per week (during dry period) .33 inches of water per week (during 100-day growing season)
Private Car Washing	90 gallons per wash (2 cars/dwelling, average once weekly)

TABLE B-2. (continued)

Public Swimming Pool	1.50 M.G. per season (10 weeks) for Olympic Pool (minimum size 75 meters x 25 yards) Estimate one pool per 1,000 families assuming 1/3 join pool membership = 330 member families per pool
Private Swimming Pool	Estimated - 20 gallons per day during summer season
Food Preparation Laundering and General Cleaning Human Consumption	Average annual demand - 51 gpcd (all in home uses) less 24 gpcd (Toilet Flushing, third item) = 27 gpcd

TABLE B-3
SUMMARY - WATER USE BY UNIT

RESIDENTIAL UNIT:

Basis:

Use per Unit - total indoor and outdoor = 416 gallons per day (summer average)

*Use per Unit - total indoor and outdoor = 827 gallons per day (maximum day 15-day period)

Persons Per Unit - 4.0

POTENTIAL REUSE: Maximum Day

	Quality* and Quality		Percent of Maximum Daily Use	
	"A-A"	"B"		
Indoor - Toilet Flushing @ 24 gpcd =		96 gpd	11.6%	
Human Consumption and Cleaning	175 gpd		21.2%	
TOTAL (indoor uses)	271 gpd			32.8%
Outdoor - Lawn Sprinkling		416 gpd	50.3%	
Car Washing		90 gpd	10.9%	
Swimming Pool		50 gpd	6.0%	
TOTAL (outdoor uses)	556 gpd			67.2%
TOTAL (ALL USES)	827 gpd			100.0%

*Reference: Linaweaver, Geyer and Wolf, "A Study of Residential Water Use"

TABLE B-4
SUMMARY - WATER USE BY UNIT

APARTMENT UNIT:

Basis:

Use per Unit - total indoor and outdoor = 202 gallons per day (summer average)

*Use per Unit - total indoor and outdoor = 414 gallons per day (maximum day - 15 day period)

Persons Per Unit - 2.5

POTENTIAL REUSE: Maximum Day

	Quantity* and Quality		Percent of Maximum Daily Use	
	"A-A"	"B"		
Indoor - Toilet Flushing @ 24 gpcd =		60 gpd	14.5%	
Human Consumption and Household Cleaning	160 gpd		38.7%	
TOTAL (indoor use)	220 gpd			53.2%
Outdoor - Lawn Sprinkling	194 gpd		46.8%	
Car Washing Swimming Pool	(assume facilities are not available to individual residence)			
TOTAL (outdoor uses)	194 gpd			46.8%
TOTAL (ALL USES)	414 gpd			100.0%

*Reference: Linaweaver, Geyer and Wolf, "A Study of Residential Water Use"

TABLE B-5
COMMERCIAL AND INSTITUTIONAL WATER USE
(and Air Conditioning Requirements)

Air Conditioning Requirements: (for all facilities)

(from"ASHRAE Guide and Data Book, Applications 1966 and 1967")

Approximate Tonnage: = 1 ton per 330 square feet of area served

Water Consumption: Q_t = Water required for cooling equipment.

Q_t = 88 gallons/ton/equivalent hour operating time

Operating Time = 800 hours/season (D.C. area)

With recirculation water consumed = $2\% \times Q_t$

Without recirculation water consumed = Q_t (wasted to sewer system)

Consider recirculation at 5 tons or greater capacity

Institutional Uses:*	(approximate only)	Gals/Capita/Day
1. Elementary Schools		16
2. Junior and Senior High Schools		25
3. Churches		3
4. Country Club	(assume shower facilities	40 (Est.)**
5. Civic Club		40 (Est.)**

Commercial Uses:* (approximate only - specific data to be considered)

1. Office Building	12 gals./100s.f. or 25 gals/Employee/Day
2. Light Industry	30 gals./Employee/Day (excluding process water)
3. Retail Sales	40 gals./Employee (excluding process water)
4. Restaurant	50 glas./Day/Seat
5. Average shopping center use**	25 gals./100 s.f./Day (including air conditioning)

* From "Community Water Systems Source Book" by Joseph S. Ameen.

** Engineer's estimate.

TABLE B-6
STUDY OF ESTIMATED WATER DEMAND
(for study area of Columbia)

I. TOTAL DEMAND (Average Annual Use) Class "A-A" = 419,440 gpd
Class "B" = 600,600 gpd

II. POPULATION SERVED:

Single Family Units - 759 units @ 4.0 persons per unit Columbia = 3,036
229 units @ 4.0 persons per unit Non-Columbia = 916
Townhouses - 1264 units @ 3.5 persons per unit Columbia = 4,424
Garden Apartments - 1132 units @ 2.5 persons per unit Columbia = 2,830
Mid-Rise Apartments - 137 units @2.5 persons per unit Columbia = 343
TOTAL POPULATION = 11,549

III. RESIDENTIAL USE: (Average Annual Use)

Domestic Use (in home) Class "A-A" = 378,000 gpd
Class "B" = 277,000 gpd
Sprinkling Use Class "B" = 241,735 gpd

IV. COMMERCIAL USE:

Shopping Center & Retail Stores:

Total Uses Class "A-A" = 8,960 gpd
Class "B" = 9,900 gpd
(includes air conditioning)

General Office Buildings:

Total Uses Class "A-A" = 2,560 gpd
Class "B" = 3,840 gpd
(includes air conditioning)

Medical Office Buildings:

Total Uses Class "A-A" = 25,900 gpd
Class "B" = 17,300 gpd
(includes air conditioning)

TABLE B-6. (continued)

V. PUBLIC SCHOOLS:

In school uses	Class "A-A"	=	3,995 gpd
	Class "B"	=	11,715 gpd
Sprinkling use	Class "B"	=	39,205 gpd

APPENDIX C

SYSTEM ANALYSIS COMPUTER PROGRAM

Chapters II and III of this report describe the methods used to analyze the local storage, treatment, and reuse system. As discussed in these chapters, the actual computations and organization of data were performed on an IBM 360 Model 65 digital computer system, using a computer program developed as a part of this study.

The main computer program performs a system analysis of each complete watershed, or of each major division of the complete watershed. Every possible geographic configuration is considered for every combination of water demands. The various system elements for each alternative are determined, sized and priced. The total costs and total benefits are computed and the net benefits are tabulated for each alternative considered. Other output information is presented, specifying the location of treatment plants, the type of treated water storage facility selected, the water demands and qualities considered, etc. In addition to the main computer program, nine subroutines are employed in the analysis. A brief description of each of these appears below.

Subroutine INTER. This subroutine is used to read the parametric tables input to the program. A linear interpolation is performed between any two points, using a forward difference technique.

Subroutine RSTORE. This subroutine calculates the cost of the least expensive permissible method of raw water storage. Input to the program includes information on the types of storage permissible (concrete tanks, steel tanks, open ponds) and on the tank locations permissible (ground level or buried).

Subroutine TREAT. This subroutine calculates the cost of the treatment plant, given the required capacity and the water quality desired.

Subroutine TSTORE. This subroutine calculates the cost of the least expensive method of treated water storage. Input to the program includes information on the types of storage permissible (concrete or steel ground level tanks, steel hydro-pnuematic tanks, standpipes, elevated tanks).

Subroutine DISTI. This subroutine calculates the cost of the distribution system, given the area to be served and the types of water demands to be met.

Subroutine COST. This subroutine computes the capital recovery factor to be applied to a capital investment to convert it to a daily cost. Input to the program includes the amortization period and the discount rate.

Subroutine TABLEO. This subroutine assembles and prints the input data in tabular form for later reference.

Subroutine OUTPUT. This subroutine assembles and prints tables of demands, total benefits, total costs, and net benefits.

Subroutine COMBI. This subroutine generates the various alternative systems for the geographical area under analysis. Each alternative is specified in terms of the portion of the total area to be served by each treatment plant and the location of the treatment plant, as well as the water quality to be produced at each plant and the demands to be connected. The subroutine also prints the final net benefit table, showing the results of the analysis for each alternative.

Following completion of the study described in this report, the computer program deck, program listing, and flow charts for the main program and each of the subroutines were turned over to the Federal Water Pollution Control Administration for their possible future use.

APPENDIX D

CONVENTIONAL STORM WATER TREATMENT SYSTEM

Chapter V of this report contains a summary description of a system for controlling storm water pollution of the Wilde Lake watershed. This system was designed by Whitman, Requardt and Associates of Baltimore, Maryland to provide a base for comparison of the various local storage, treatment and reuse systems.

This appendix contains a verbatim copy of the Whitman, Requardt and Associates report on the Conventional Treatment System and provides additional details on the basis for the design of this system which were not included in Chapter V.

SYSTEM CONCEPTUAL DESIGN

A. Purpose

The purpose of the program is to collect and treat storm water in the Wilde Lake watershed of Columbia, Maryland to control the pollution in Wilde Lake and the stream below it resulting from storm drainage runoff. The "Conventional System" of pollution control will employ the conventional storm drain inlet and piping systems of Columbia leading to interceptors, pumping stations, and a centralized treatment basin where the runoff will be treated prior to discharge to the Lake.

This system is intended to provide a comparison to the "Local Storage System," a concept which utilizes numerous treatment basins, each local to a sub-drainage area of Columbia for which the added feature of storm water reuse has been considered. For purposes of comparison it was important to work from consistent criteria in the analysis and design of both systems, and close coordination has been maintained with personnel of Hittman Associates in this regard.

B. Requirements

1. Area Considered

The drainage area tributary to Wilde Lake (See Figure V-1, page V-3) was selected for the program. This included all areas whose runoff naturally contributes to Wilde Lake.

2. Water Quality

The water quality criteria and standards of the Department of Water Resources, State of Maryland, relating to "Water Contact Recreation" were chosen as the minimum quality of water acceptable for the discharge into the Lake. This is equivalent to Class "C" water previously described in Chapter II, Paragraph D.3.

3. Water Quantity

Determination of the quantities of runoff to be handled and treated was based on the frequency and the magnitude of runoff versus economics and space requirements of collection and treatment. For instance, the storage area required to hold and treat the runoff from a ten year storm is not available in the allotted open spaces around the Lake. Therefore, based on storage areas available and practical sizing of interceptor systems, wet wells, and pumping stations (See Figure V-1, page V-3), a one year storm was selected for design.

To compute the quantity of probable runoff, hydrographs for the sub-drainage areas were synthesized, and runoff curves for one year, five year, and ten year storms were developed for a 60 minute storm as a function of the imperviousness of the areas. Peak volume is a more important parameter than peak rate for the purposes of this study. Thus, the 60 minute rainfall was selected as the length of storm which would approach the maximum volume of runoff for each area selected. These curves are shown on Figure D-1 and form the basis for computing the design flows.

4. Treatment

The principal problems in treatment of storm water are the removal of Suspended Solids and the reduction of the Biochemical Oxygen Demand (BOD). To achieve the required quality of the discharge, sedimentation, filtration, and disinfection (chlorination) were chosen as the treatment to be applied.

C. System Description

1. General

The object of this plan is to collect the storm water from up to a one year storm and deliver it to the treatment basin for settlement, filtration, and chlorination.

The system naturally divides itself into two basic areas. The first, designated Area "A" on Figure V-1, consists of 954 acres, the runoff from which flows by gravity to the treatment basin. The second, designated Area "B" on Figure V-1, is that area (165 acres) whose runoff must be pumped to the treatment basin.

Parts of Area "A" and all of Area "B" are downstream from the treatment basin, and for this area, a system consisting of pumping stations, force mains, gravity interceptors, and a holding pond must be developed to divert the runoff to the basin for treatment.

2. Flows

Utilizing the one year and ten year curves from Figure D-1 flows were developed for the area tributary to the holding pond in Area "B", for the areas contributing to the pumping stations by gravity, and for the area contributing directly to the treatment basin, i.e., Area "A". These areas and the resulting flows are shown in Table D-1.

3. Holding Pond

A reconnaisance of the area was made for feasible holding pond locations, reasoning that damping of peak runoff rates could best be accomplished by storing the large volume of runoff created by the heavy storms in natural ponds. Controlled effluent from such ponds minimuzes the size of the collecting sewers, pumping stations, and wet wells required to transport the flows to the treatment basin.

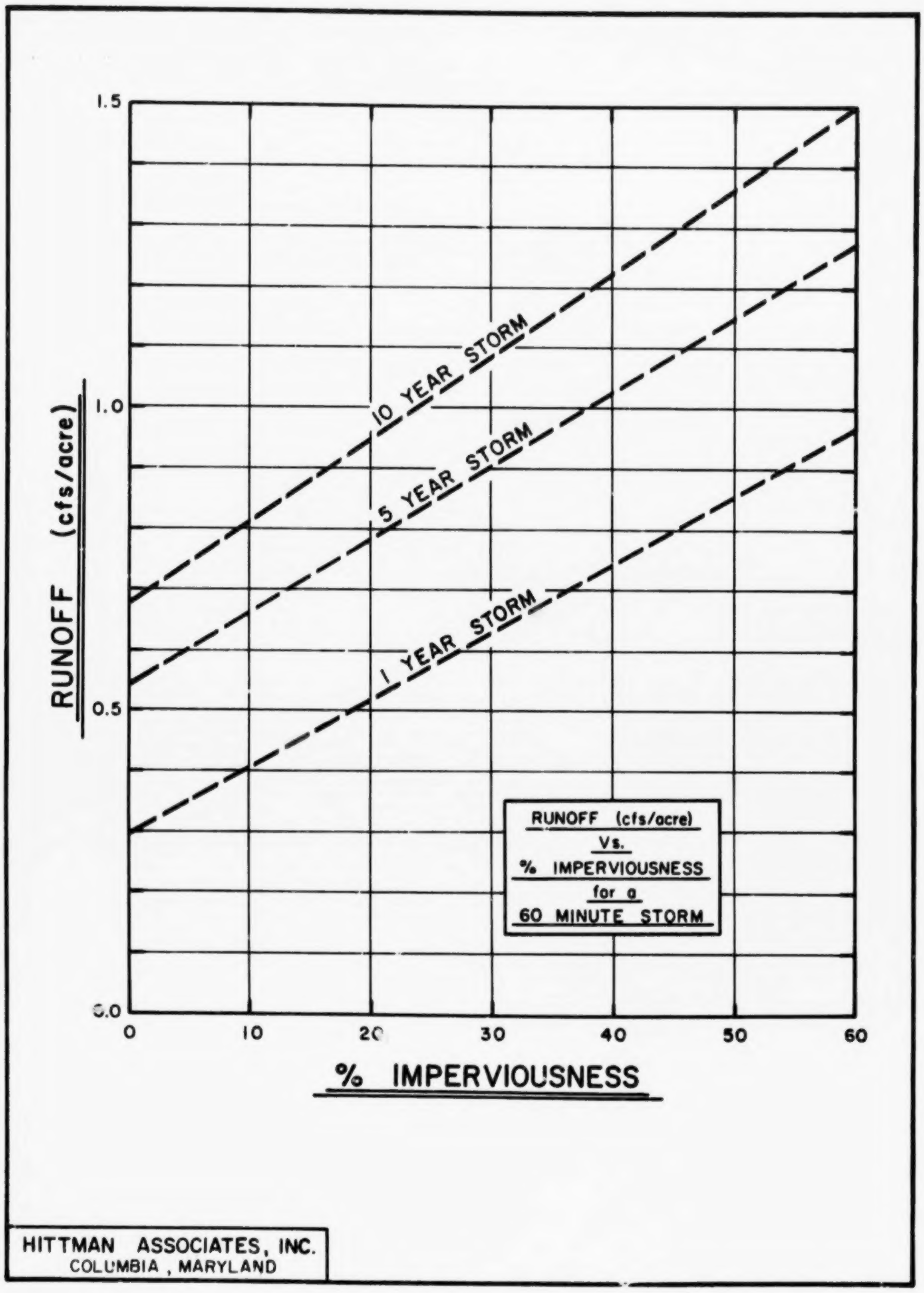
1.5
1.0
0.5
0.0
0
10
20
30
40
50
60
RUNOFF (cfs/acre)
% IMPERVIOUSNESS
10 YEAR STORM
5 YEAR STORM
1 YEAR STORM
RUNOFF (cfs/acre)
Vs.
% IMPERVIOUSNESS
for a
60 MINUTE STORM
HITTMAN ASSOCIATES, INC.
COLUMBIA, MARYLAND

Figure D-1.

TABLE D-1. TRIBUTARY AREAS AND FLOWS

Description	Area (Acres)	1 Year Storm (MG)	10 Year Storm (MG)
1. Tributary to Holding Pond	89	1.84	2.97
2. Tributary to Pumping Station #1	10	0.20	0.33
3. Tributary to Pumping Station #2 (other than flow from Pumping Station #1)	40	0.64	1.06
4. Tributary to treatment basin by gravity (Area "A")	954	13.35	24.40
5. Fringe areas of Wilde Lake not collected	26	0.47	0.86
TOTAL TO WILDE LAKE	1119	16.50	29.62

In Area "B", however, only one suitable location for a holding pond was found to be available. This pond is shown on Figure V-1, page V-3. The total area contributing to the holding pond was developed beyond the natural tributary area by the addition of an interceptor system extending west into Bryant Woods so that maximum advantage could be derived from the effect of the storage. Storage requirements for this area for a one year storm are 1.84 MG as shown in Table D-1.

In order to assure an empty basin to store each storm the effluent piping and pumping facilities were designed to drain the pond (1.84 MG) in no longer than a one day period. This allows for the occurrence of a one year storm on two consecutive days without an overflow of the pond.

Provisions have been included to overflow runoff in excess of the storage capacity of the pond directly to Wilde Lake.

4. Pumping Station #1

The use of a gravity outfall sewer between the holding pond and the treatment basin was studied but found to be impossible because of distance and relative elevations. Also, a gravity sewer from the pond to Pumping Station #2 was found to be impractical because of lack of gradient. To effect a practical solution Pumping Station #1 was placed to drain the holding pond and the other areas tributary to the lake at the station location. (See Figure V-1, page V-3) A pumping capacity of 2.5 MGD was selected for this station to handle the one year storm (1.84 MG) and to reduce the wet well size.

The force main for this system, shown in Figure V-1, discharges to the upper end of the gravity interceptor placed to collect the remainder of Area "B" located south of the lake.

Provisions for overflow to the lake should be incorporated in the pumping station for flows exceeding design capacity and for emergencies.

5. Gravity Interceptors

Gravity interceptors have been strategically placed to:

(a) Divert the maximum amount of runoff directly to the treatment basin

(b) Divert the maximum amount of runoff in Area "B" to the holding pond

(c) Collect from the maximum area along the fringes of Wilde Lake

The principal object of interceptors described in items a and b above is to reduce the maximum rate runoff to Pumping Stations #1 and #2 from areas directly tributary to them and thereby minimize the required volume of the wet wells.

Figure V-1, page V-3 shows these interceptors and their relationships with the treatment basin, the holding pond, and Pumping Stations #1 and #2.

6. Pumping Station #2

Pumping Station #2 will deliver the one year storm from Area "B" to the treatment basin for processing. In addition to the flow from Pumping Station #1, the wet well will receive 0.64 MG from the interceptors described in item c above. A large percentage of this flow arrives in a relatively short time, and to assure holding this water, a very large wet well is required.

Dual 5 MGD pumps are required for the station to deliver the one year storm runoff to the treatment basin in no longer than 24 hours. The 10 MGD maximum capacity (both pumps running) allows some reduction in the size of the wet well.

Runoff from this area resulting from storms larger than the one year storm will overflow to the lake.

7. Treatment Basin

a. General. The treatment basin is located at the head of Wilde Lake on the main stream draining the area. The location is within the natural open spaces designated by the development plan. The operating levels of water in the basin and the basin size are limited by the close proximity of the developed areas.

The treatment basin, shown in detail on Figure V-2, page V-4, consists of an earth dam and an excavated basin to provide some silt storage and runoff storage large enough to process the one year design storm. The basin shown represents the most practical design within the limits imposed by the geography of the area, property boundaries, and aesthetic considerations. Figure D-2 shows the flood routing relationships of a one year storm on the basin as well as a plot of water level elevations during the design storm. From this plot it may be seen that the design storm does not overtop the spillway and is completely processed through the basin before entering Wilde Lake.

Essential features of the basin are:

(1) Dam height - 15 feet from existing stream bed to top of dam elevation 349.0

(2) Spillway length - 60 feet

(3) Spillway overflow elevation - 344.5

(4) One year storm maximum water level - 344.0

(5) Normal pool storage - 2.6 MG

(6) Maximum storm storage - 12.4 MG from normal pool to spillway overflow

(7) High water level - 346.0 (10 year storm)

(8) High water volume - 16.6 MG from normal pool to high water level

(9) Area at high water level - 424,000 square feet (9.75 acres)

(10) Maximum water level for dam design - 346.8 (100 year storm)

b. Sedimentation. Removal of suspended solids and reduction of turbidity in storm water are functions of water surface area, storage volume, and overflow rate. An overflow rate of approximately 100 gpd per square foot of reservoir area, for example, will settle particles down to 10 macron size or, essentially, silt. The water surface area and overflow rate of the treatment basin are variable with the water level elevation which depend on actual volume of water in the basin.

Table D-2 shows values of Q and area for various elevations and the size particles that will be settler out under each set of conditions. It may readily be seen that the treatment basin will settle particles in the range of

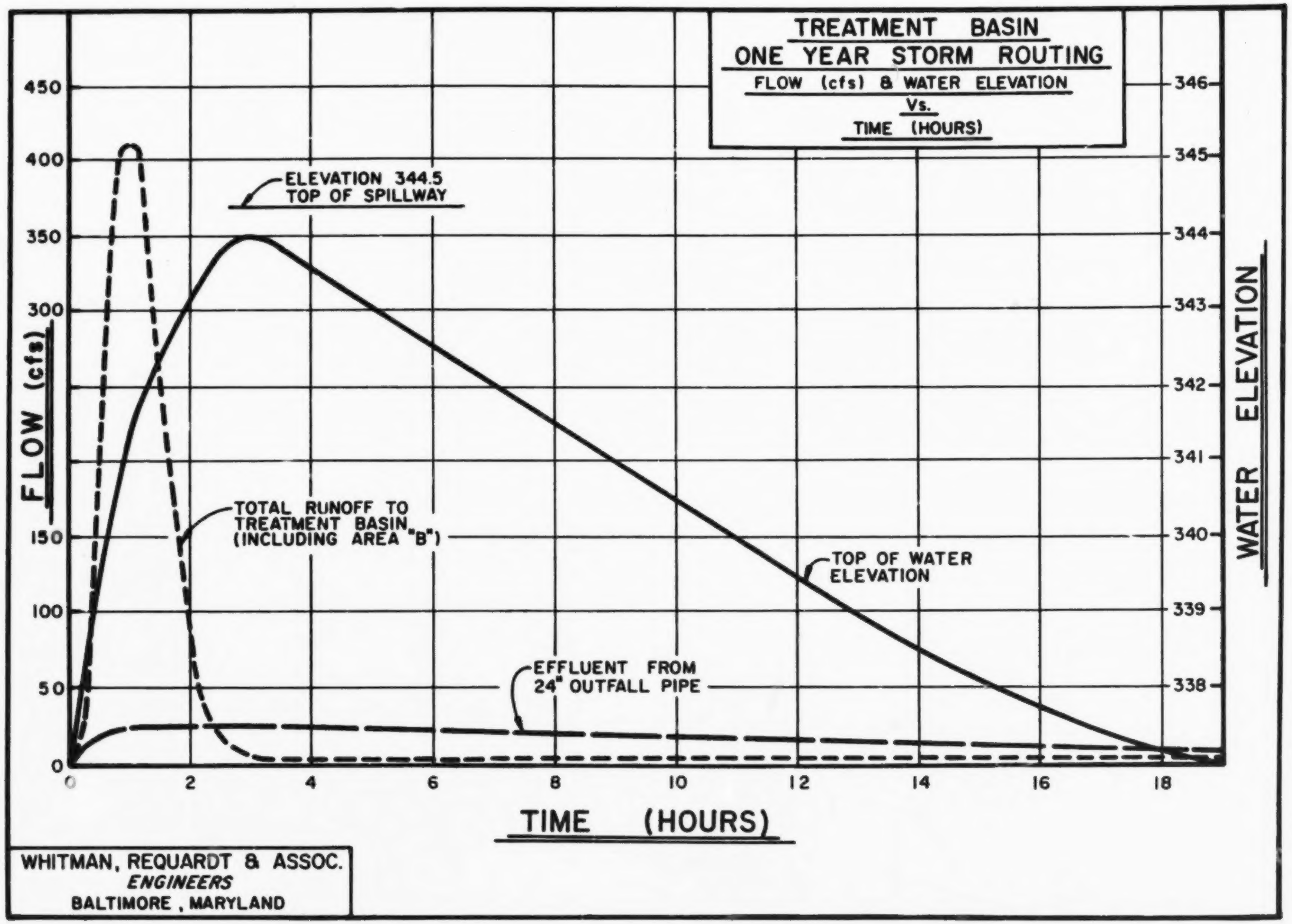
TREATMENT BASIN
ONE YEAR STORM ROUTING
FLOW (cfs) & WATER ELEVATION
Vs.
TIME (HOURS)
ELEVATION 344.5
TOP OF SPILLWAY
TOTAL RUNOFF TO
TREATMENT BASIN
(INCLUDING AREA "B")
TOP OF WATER
ELEVATION
EFFLUENT FROM
24" OUTFALL PIPE
FLOW (cfs)
WATER ELEVATION
TIME (HOURS)
450
400
350
300
150
100
50
0
346
345
344
343
342
341
340
339
338
0
2
4
6
8
10
12
14
16
18
WHITMAN, REQUARDT & ASSOC.
ENGINEERS
BALTIMORE, MARYLAND

Figure D-2

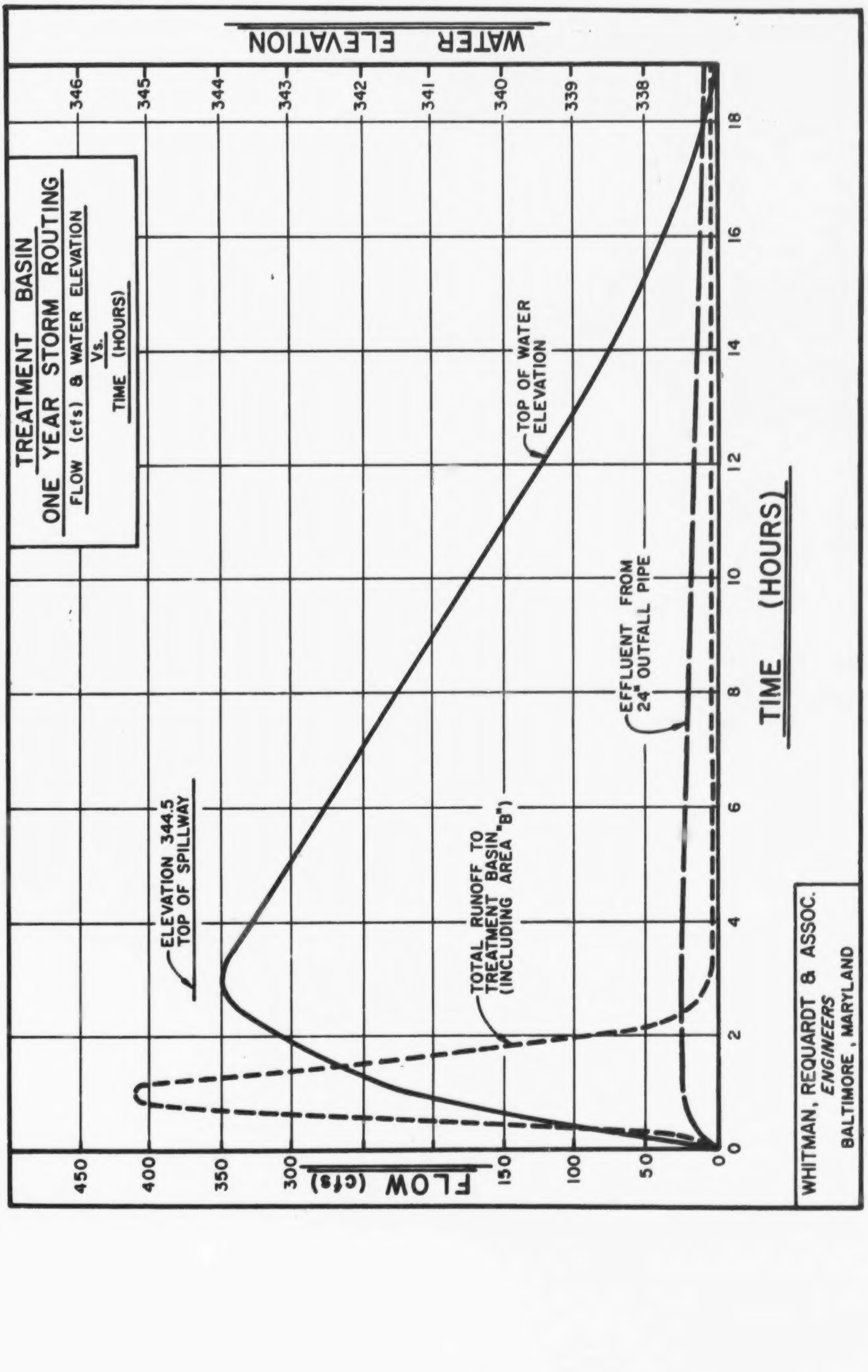
TREATMENT BASIN
ONE YEAR STORM ROUTING
FLOW (cfs) & WATER ELEVATION
Vs.
TIME (HOURS)
FLOW (cfs)
0
50
100
150
200
250
300
350
400
450
WATER ELEVATION
338
339
340
341
342
343
344
345
346
TIME (HOURS)
0
2
4
6
8
10
12
14
16
18
ELEVATION 344.5 TOP OF SPILLWAY
TOTAL RUNOFF TO TREATMENT BASIN (INCLUDING AREA "B")
EFFLUENT FROM 24" OUTFALL PIPE
TOP OF WATER ELEVATION
WHITMAN, REQUARDT & ASSOC.
ENGINEERS
BALTIMORE, MARYLAND

Figure D-2

TABLE D-2. PARTICLE REMOVAL, SURFACE AREA AND OVERFLOW RATES

Water Elevation	Effluent Q (MGD)	Water Surface Area (square feet)	Overflow Rate (gal/day/sf)	Size of Particles Removed (Microns)
334.0	10.0	181,000	55	7.5
338.0	14.0	186,000	75	8.8
340.0	19.5	196,000	100	10.1
342.0	24.0	219,000	110	10.6
344.5	28.5	320,000	89	9.5
346.0	278.0	424,000	660	26.0

silt (approximately 10 microns) for overflow rates up to the spillway (elev. 344.5) which includes the one year design storm (elev. 344.0). For the 10 year storm high water level of 346.0 with water going over the spillway, however, only 26 micron particles (fine sand) will be settled out.

Class "C" water requires greater reduction of turbidity and settlement of particles smaller than 10 microns. It may then be concluded that the treatment basin, although satisfactory as a primary sedimentation basin, will not satisfy the design criteria, i.e., produce Class "C" water. Further treatment is called for by means of filtration.

c. Gravel Filter. The gravel filter shown in Figure V-2, page V-4 constructed of 20 feet of fine, uniform pea gravel, backed by a rock dam. The flow through the filter is a function of the size of the gravel and the thickness of the filter. Particle removal is dependent on flow rate and filter size. To attempt to process the heaviest flows of water and remove all of the turbidity would require a filter of such magnitude and expense as to be impractical from the standpoint of maintenance and cost. Therefore, a filter was selected to give the best practicable results consistent with the criteria.

The volume of the basin up to the top elevation of the filter coupled with the flow of water through the filter during a storm allows up to approximately 5 MG of water to be processed and stored prior to overtopping of the filter. Thus, water from the larger storms will pass over unfiltered. However, the first 5 MG of the water from any storm will be filtered and since it is general knowledge that the initial runoff from an area contains the type of solids and debris which pose the greatest threat to pollution, the filter as planned will effectively function over the greatest percentage of time and is predicted to effectively produce Class "C" water. Reference to Figure D-3 shows that a storm of 5 MG (7.75 cfs-days) magnitude is exceeded only six percent of the time.

The efficiency of the filter will, of course, diminish as its permeability is decreased by the filtered solids. This becomes a problem of periodic maintenance similar to the removal of silt from the sedimentation

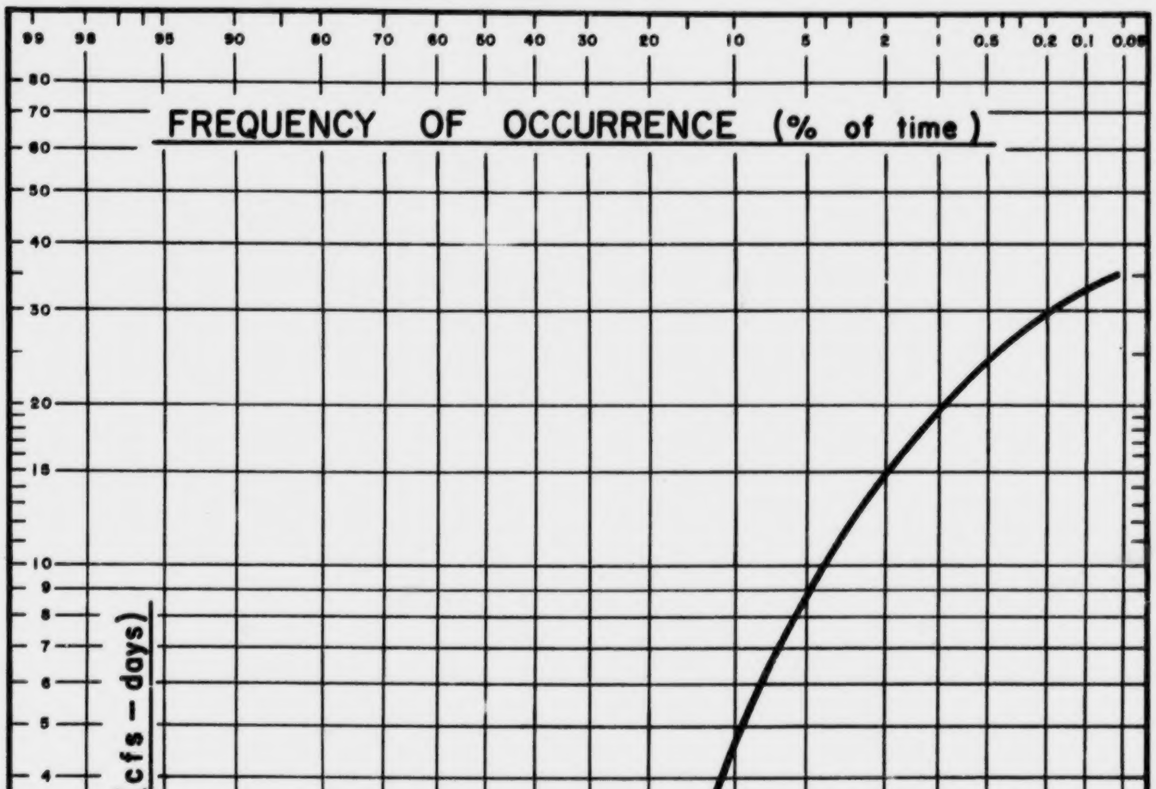
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
0.05
FREQUENCY OF OCCURRENCE (% of time)
80
70
60
50
40
30
20
15
10
9
8
7
6
5
4
3
2
1.5
1.0
.9
.8
.7
.6
.4
.3
.2
RUNOFF (cfs - days)
RUNOFF (cfs-days)
Vs.
FREQUENCY OF OCCURRENCE %
for
STORMS RECORDED 1960-1965
WHITMAN, REQUARDT & ASSOC.
ENGINEERS
BALTIMORE, MARYLAND

Figure D-3.

basin. The filter is thick enough to allow several scrapings for removal of the clogged layer prior to complete replacement. Without demonstrated results from such a basin filter arrangement it is difficult to establish the exact nature of the maintenance and operation. However, it is assumed that once all upstream development is complete, the front face of the filter must be removed approximately once every year and the entire filter must be replaced approximately every ten years. While development continues upstream, unless temporary silt basins are installed, more frequent maintenance will probably be required.

d. Outlet Structure. The outlet structure consists mainly of an overflow weir, outlet structure and conduit, and chlorination equipment. The stated objective is to process a one year storm and empty the basin in one day. During the design storm the basin receives 13.35 MG from Area "A" plus the water pumped up from Area "B" downstream (2.68 MG). This means that the outlet structure must pass a total of 16.03 MG. The 24 inch outlet conduit selected will drain the basin in 19 hours at an average flow of 20 MGD with the rate varying from 27.5 MGD at the high water level elevation to 0.5 MGD flow at low pool.

The chlorination equipment required to adequately disinfect the effluent will, of course, have to provide variable dosages to meet the range of outflow rates. It is anticipated that the feed may be operated from a float control providing five to seven different rates. The actual amount of chlorine required to meet the requirements of Class "C" should be based upon actual conditions and observed results. It may be assumed that a dosage of three to five ppm is realistic.

The normal pool elevation of the treatment basin is maintained by stop logs in the outlet structure. The pool may be drained for silt removal or any other purposes by removal of the logs.

D. Summary

From the foregoing discussion, it may be concluded that the system will collect, process, and treat the one year design storm and deliver Class "C" water to Wilde Lake. The holding pond and the pumping station wet wells have been kept to a minimum by oversizing the pumps and by continuous operation throughout the storm's duration. The wet wells will not store the runoff from a one year storm unless the pumps are in operable condition, but the probability of a breakdown occurring during the time of need is minimal. The penalty for pump breakdown is merely overflow to Wilde Lake and this does not justify the additional expense for total wet well storage or additional pumps.

The treatment basin is well within the storage volume requirements for the design storm. In fact, an additional volume of water could be stored and processed by the basin without overtopping the spillway.

To examine the performance characteristics of the basin and the effect of spillway overflow on Wilde Lake, a probability curve has been created for the five years recorded from the synthesized runoff record. This curve is shown on Figure D-3, and represents storms on the entire drainage area selected.

A storm of 18.6 MGD on the entire drainage area will just utilize the 12.4 MG storage of the basin without overtopping the spillway. Examination of the probability curve, Figure D-3, reveals that for the five years of recorded storms a runoff in excess of 18.6 MGD (28.9 cfs-days) was exceeded only 0.25 percent of the time. Comparisons with the recorded storms show that this runoff was exceeded only five times in the five years recorded and that the actual amount of water over the spillway would be 6.3 MG which is 0.27 percent of the total amount of runoff to the lake during this period. It may, therefore, be concluded that the effect of these limited volumes of water overflowed to Wilde Lake (capacity approximately 45 MG) will be minimal.

CONCEPT EVALUATION

Construction costs for the Conventional System are based on early 1968 construction unit prices developed by updating recorded construction prices for similar types of work. These prices represent an estimate of the total contractor's bid including his direct costs, overhead, and profit.

To arrive at the project costs it was necessary to increase the construction costs by the following percentages to reflect additional charges experiences in this type of construction:

Construction Contingencies	15%
Project Costs (Engineering, Legal Administrative, etc.)	20%
Escalation (per year)	5%
TOTAL INCREASE TO BASIC CONSTRUCTION COST	40%

The total annual costs were developed for each plan by estimating the annual debt service and adding this to annual maintenance and operating charges. The annual debt service is estimated by amortizing the construction cost at 3.2 percent* over thirty years.

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*Note: Interest rate and amortization period provides by Hittman Associates, Inc.

A. CONSTRUCTION COST :

1.	Holding Pond :		
	Site Preparation	$ 2,000.	
	Excavation	$ 17,000.	
	Outlet Structure	$ 5,000.	
	TOTAL	$ 24,000.	$ 24,000.
2.	Pumping Station #1 :		
	Wet Well (70,000 gallon)	$ 65,000.	
	Package Pumping Station (5 MGD)	$ 50,000.	
	Electrical	$ 2,000.	
	TOTAL	$117,000.	$117,000.
3.	Force Main :		
	16" Pipe	$ 30,000.	
	TOTAL	$ 30,000.	$ 30,000.
4.	Interceptor Systems :		
	To Holding Pond	$ 50,000.	
	To Pumping Station #1	$ 22,000.	
	North Side to Pumping Station #2	$ 58,000.	
	South Side to Pumping Station #2	$ 22,000.	
	To Treatment Basin	$ 53,000.	
	TOTAL	205,000.	$205,000.
5.	Pumping Station #2 :		
	Structural (incl. .37 MG wet well)	$200,000.	
	Mechanical and Electrical	$ 80,000.	
	Force Main and Outfall Sewer	$ 20,000.	
	TOTAL	$300,000.	$300,000.
	SUB TOTAL - Items this page		$676,000.

SUB TOTAL brought forward from Page D-14			$676,000.
6. Treatment Basin :			
Outlet Structure			
Architectural	$ 2,500.		
Structural	$ 10,000.		
Mechanical	$ 10,000.		
Electrical	$ 4,500.		
	$ 27,000.	$ 27,000.	
Dam			
Spillway Structure (incl. walls & Slabs)	$ 45,000.		
Embankment	$ 10,000.		
	$ 55,000.	$ 55,000.	
Filter			
Pea Gravel	$ 6,000.		
Rock Dam	$ 12,000.		
	$ 18,000.	$ 18,000.	
Excavation			
Site Preparation	$ 5,000.		
Treatment Basin	$150,000.		
Channel Improvement	$ 8,000.		
	$163,000.	$163,000.	
	TOTAL	$263,000.	$263,000.
TOTAL CONSTRUCTION COST			$939,000.
Add 40% for Construction Contingencies, Project Costs, and Escalation for one Year			$375,500.
TOTAL PROJECT COST			$1,314,500.

B. MAINTENANCE AND OPERATION COSTS :

1.	Removal of sediment in holding pond and treatment basin bi-annually		$ 5,000./year
2.	Depreciation		$ 8,000./year
3.	Gravel filter maintenance and replacement		$ 1,000./year
4.	Erosion repair		$ 1,000./year
5.	Routine attention to equipment, including minor repairs and maintenance (one man part time)		$ 6,000./year
6.	Routine care and maintenance of grounds, trash removal from filter, and algae control (one man part time)		$ 4,000./year
7.	Transportation, tools, and equipment		$ 3,000./year
8.	Cost of chlorine		$ 1,200./year
9.	Pumping cost		
	Demand charge	$ 3,000.	
	Energy use charge	$ 800.	
		$ 3,800.	$ 3,800./year
	TOTAL YEARLY COST OF MAINTENANCE AND OPERATION		$ 33,000.

C. ANNUAL COSTS :

1.	Debt service	$ 67,000.
2.	Maintenance and operation	$ 33,000.
	TOTAL ANNUAL COSTS	$100,000.

APPENDIX E

PUBLIC ACCEPTANCE AND LEGAL ASPECTS OF THE REUSE OF STORM WATER

In order to determine legal aspects of the reuse of storm water, particularly in sub-potable form and through dual distribution systems, a survey of applicable public codes and regulations was made. This work was performed by Whitman, Requardt and Associates and a complete report was received from them of their investigation of **16** different codes and sets of regulations. In the case of state and local codes, the agencies concerned were contacted by Whitman, Requardt and Associates and interpretations of the codes sought for the storm water reuse application. No conflicts with existing regulations were found, although several codes contain provisions that would affect the design of a dual distribution system. The following table summarizes the results of this investigation.

TABLE E-1

REVIEW OF PUBLIC CODES

Index of Regulations

NO.	TITLE	DESCRIPTION	SUMMARY
A	MARYLAND STATE DEPARTMENT OF HEALTH -	**43L06** State Plumbing Code	The State Plumbing Code does not appear to contain any regulations that would prohibit or conflict with the construction of separate potable and non-potable water systems within a dwelling. The code includes regulations concerning cross-connections and physical requirements for water service connections.
B	MARYLAND STATE DEPARTMENT OF HEALTH -	**43L02** Regulations Governing Individual Water Supply and Sewage Disposal Systems - Where Public Systems are not Available	These regulations do not appear to prohibit the construction of separate potable and non-potable systems.
C	MARYLAND STATE DEPARTMENT OF HEALTH -	**43L03** Regulations Governing Water Supply and Sewerage Systems in the Subdivision of Land in Maryland	No apparent conflict.
D	NATIONAL PLUMBING CODE	by American Society of Mechanical Engineers	Prohibits use of non-potable water in fixtures which are accessible for drinking or culinary purposes. No apparent conflict in other regulations.
E	BALTIMORE COUNTY PLUMBING CODE		Provides for use of non-potable water for toilet flushing and in other fixtures not accessible for drinking and culinary purposes. Requires color-coding of non-potable water lines.
F	BALTIMORE COUNTY STANDARD SPECIFICATIONS FOR CONSTRUCTION, DEPARTMENT OF PUBLIC WORKS		No apparent conflict
G	BALTIMORE COUNTY DESIGN MANUAL		No apparent conflict
H	BALTIMORE COUNTY STANDARD DETAILS		No apparent conflict

TABLE E-1. (Continued)

NO.	TITLE	DESCRIPTION	SUMMARY
I	MARYLAND STATE DEPARTMENT OF HEALTH	43L05 Regulations Governing Submission of Plans for Approval and Record of Systems of Water Supply, Sewerage, Storm Water, and Refuse Disposal	No apparent conflict
J	ANNE ARUNDEL COUNTY DESIGN MANUAL		No apparent conflict
K	PUBLIC SERVICE COMMISSION	Requirements Prior to Authorization Before Commencing Construction	No apparent conflict
L	PUBLIC SERVICE COMMISSION	Regulations Governing Service Supplied by Water Companies	No apparent conflict
M	HOWARD COUNTY SUBDIVISION		No apparent conflict
N	HOWARD COUNTY STANDARD DETAILS FOR CONSTRUCTION		No apparent conflict
O	HOWARD COUNTY PLUMBING CODE		Howard County reports that the Maryland State Plumbing Code and the ASME National Plumbing Code are used in the County.
P	FEDERAL HOUSING ADMINISTRATION	Minimum Property Requirements	The FHA requires adherence to the ASME National Plumbing Code.

APPENDIX F

DEMONSTRATION SYSTEM PRELIMINARY DESIGN

As stated in Chapter IV, Local Storage, Treatment, and Reuse System Conceptual Design, preliminary designs for the facilities for the demonstration program were prepared to serve several purposes. First, these drawings, the associated system descriptions, and the cost estimates were used as the basis for the planning of the demonstration program discussed in Chapter VII of the report. Secondly, these drawings are used to illustrate the construction details of the typical storage reservoirs, and pretreatment units that would be used with the various local systems for collection, storage, treatment, and reuse of storm water which are described in Chapter IV. Thirdly, as stated in Chapter IV, the drawings prepared for the demonstration system were also used to make unit and materials estimates which were used as the basis for estimating the costs of the overall system.

The preliminary design of the demonstration system was prepared by Whitman, Requardt and Associates of Baltimore, Maryland. This appendix contains a complete copy of their report on the facilities for the demonstration program. In addition to the storage reservoirs and pretreatment units noted above, this report contains information on the gaging and sampling stations, the treatment plant, and the reuse systems for the various demonstration program approaches discussed in Chapter VII. The treatment plant discussed in this appendix is one for general test purposes and for the evaluation of various treatment processes for storm water. Accordingly, it would not be typical of the treatment plants that would be used with future local storage and treatment systems. After the effectiveness of various processes were evaluated as part of the demonstration program, it is envisioned that packaged treatment plants would be manufactured specifically for the treatment of storm water. These would be installed either in underground vaults, surface structure, or behind patio type fencing materials depending upon the requirements of the specific site. Appendix H contains a flow chart and layout of a typical existing packaged treatment plant that could be adapted for the treatment of storm water.

DEMONSTRATION SYSTEM

Preliminary Design

A. Purpose

The purpose of this program is to demonstrate the concepts for control of stream pollution due to storm drainage runoff discussed previously in this report. The demonstration system will utilize three storm retention ponds with settling facilities and chlorination to treat the storm water prior to discharge back to the streams.

Two gaging and sampling stations will be constructed to provide a comparison between the treated water and untreated water from an area of approximately equivalent characteristics.

The added feature of reuse of storm water for domestic purposes has been incorporated in the program. A treatment plant will be attached to one of the retention ponds to demonstrate the production of various classes of usable water. A portion of the plumbing system of a nearby group of apartment buildings will be connected to utilize this water for some domestic purposes.

B. Requirements

1. Area Considered

The south-westernmost portion (285 acres) of the Wilde Lake drainage area (See Figure F-1) was selected for the demonstration program. An equivalent area (330 acres) just to the north was selected as the control area.

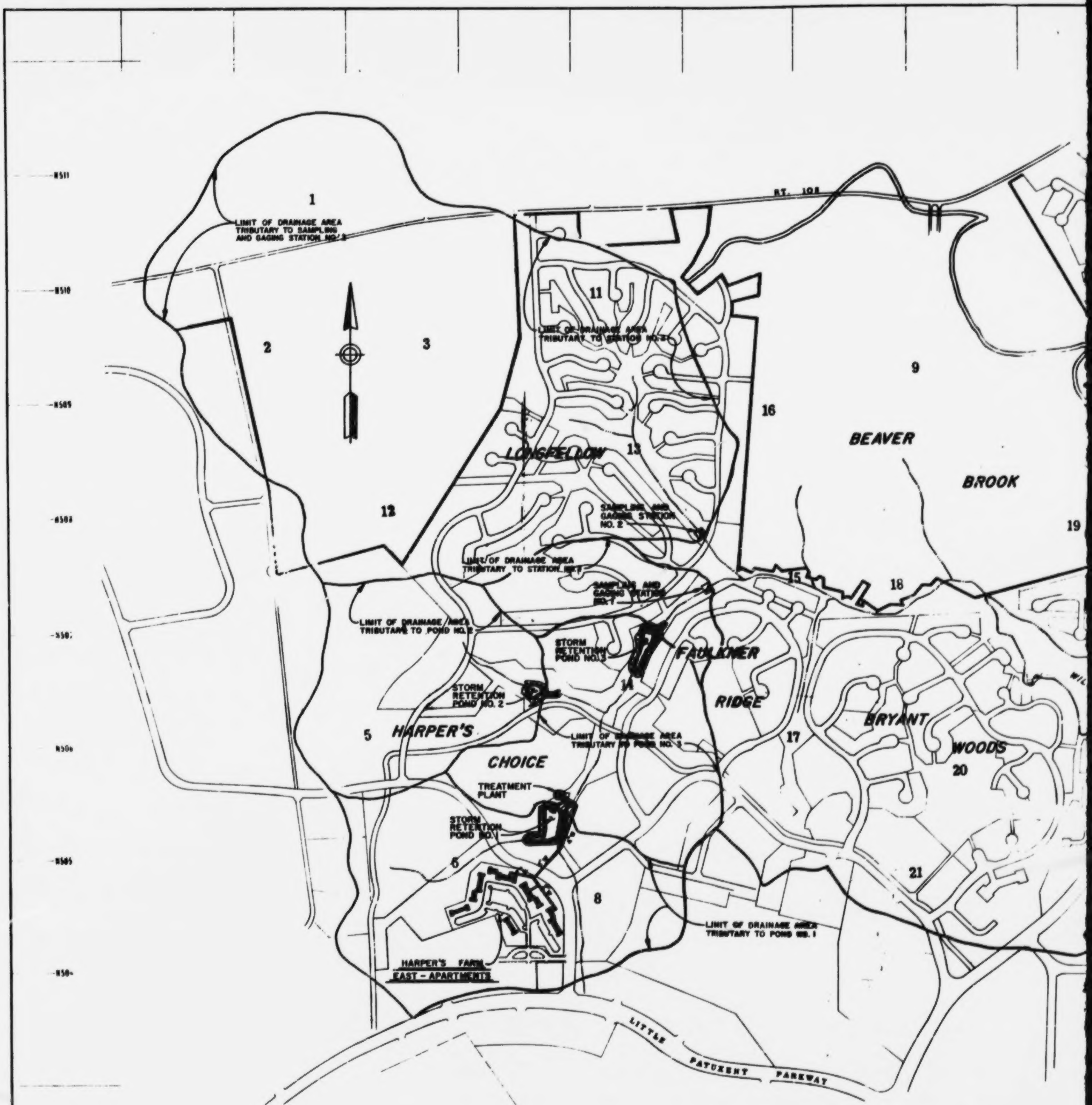
2. Water Quality

The water quality criteria and standards of the Department of Water Resources, State of Maryland, relating to "Water Contact Recreation" were chosen as the minimum quality of water acceptable for discharge to the streams. This is equivalent to Class "C" water described in Section II, Paragraph D-3 of this report.

The treatment plant will be capable of producing Class "B", Class "A", and Class "AA" water as described in the same Section and Paragraph.

3. Water Quantity

A one year storm was selected as the design flow to be retained and treated in order to remain consistent with other portions of the project.


N511
N510
N509
N508
N507
N506
N505
N504
RT. 108
1
2
3
LIMIT OF DRAINAGE AREA TRIBUTARY TO SAMPLING AND GAGING STATION NO. 3
11
LIMIT OF DRAINAGE AREA TRIBUTARY TO STATION NO. 2
9
16
BEAVER
BROOK
19
LONGFELLOW
13
12
SAMPLING AND GAGING STATION NO. 2
LIMIT OF DRAINAGE AREA TRIBUTARY TO STATION NO. 1
15
18
SAMPLING AND GAGING STATION NO. 1
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 2
STORM RETENTION POND NO. 3
FAULKNER
14
RIDGE
STORM RETENTION POND NO. 2
BRYANT
5
HARPER'S
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 3
17
WOODS
20
CHOICE
TREATMENT PLANT
STORM RETENTION POND NO. 1
6
21
8
LIMIT OF DRAINAGE AREA TRIBUTARY TO POND NO. 1
HARPER'S FARM EAST - APARTMENTS
LITTLE PATUXENT PARKWAY
831
832
833
834
835
836
837
838

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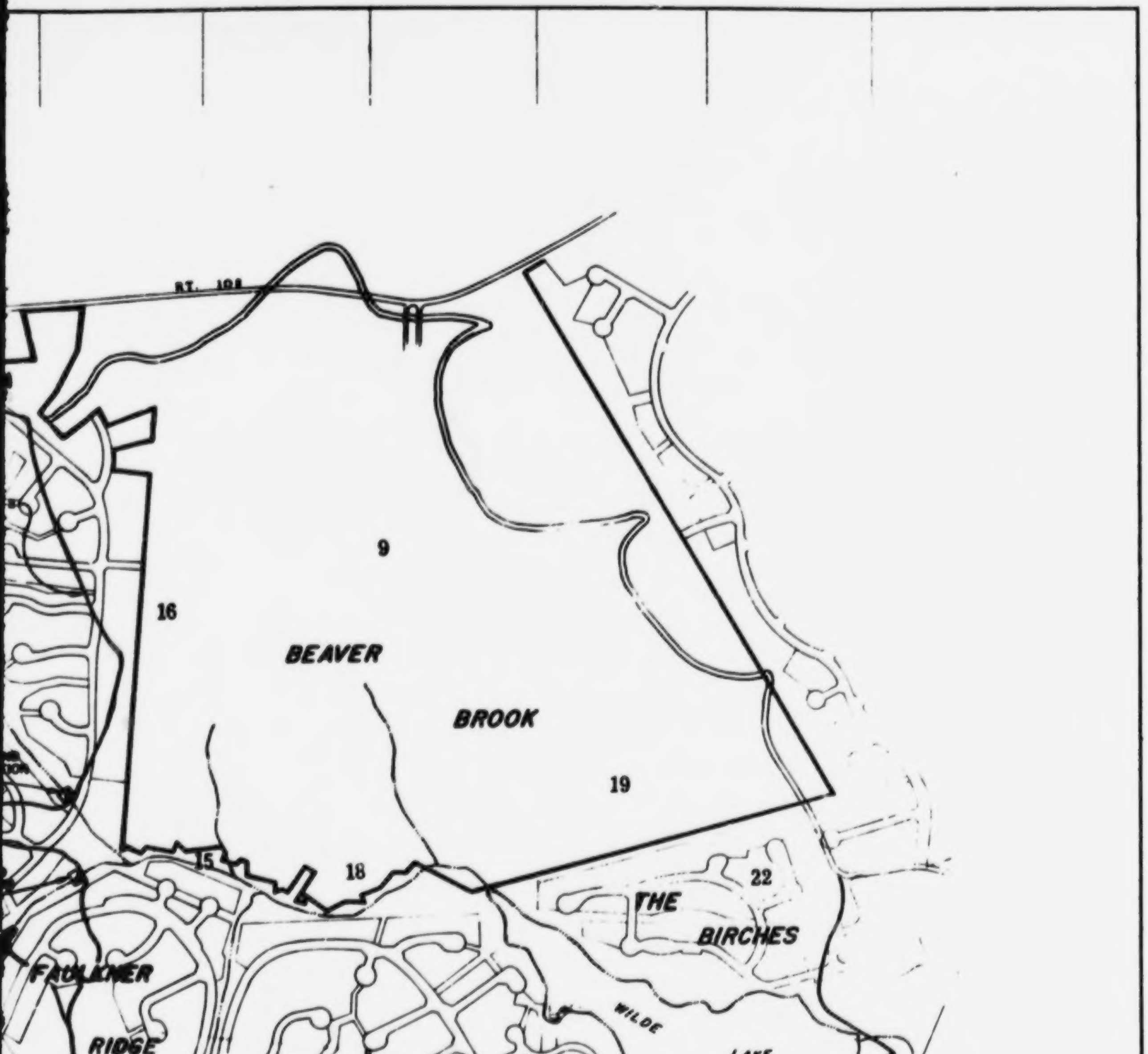
A


RT. 108
9
16
BEAVER
BROOK
19
15
18
22
THE
BIRCHES
FAULKNER
RIDGE
WILDE
LAKE
BRYANT
WOODS
17
20
21
AREA
NO. 1
LIMIT OF DRAINAGE AREA
TRIBUTARY TO POND NO. 1
PATUXENT
PARKWAY
FWPCA Contract No. 14-12-20
WILDE LAKE WATERSHED
Columbia, Md.
LOCATION PLAN
FOR
DEMONSTRATION PROGRAM
SCALE: 1" = 400'
835
836
837
838
839

B

Figure F-1

To compute the quantity of probable runoff for the storm retention ponds, hydrographs for the subdrainage areas were synthesized, and runoff curves for one year, five year, and ten year storms were developed for a 60 minute storm as a function of the imperviousness of the areas. Since peak volume is the important parameter rather than peak rate with respect to the retention ponds, the 60 minute rainfall was selected as the length of storm which would approach the maximum volume of runoff for each area selected. These curves are shown in Appendix D, Figure D-1, and form the basis for computing the design volumes for the retention pond.

For the sampling and gaging stations, on the other hand, peak flow is the more important parameter for design, since measurement rather than impoundment and treatment of the water is desired. For this requirement, the rational method of design ($q = CIA$) was utilized to determine these peak rates.

4. Treatment

The principal problems in treatment of storm water are the removal of suspended solids and the reduction of the Biochemical Oxygen Demand (BOD). To achieve the required quality of water for discharge to the streams, sedimentation, both by natural and mechanical means, and disinfection (chlorination) were chosen as the treatment to be applied.

To achieve the required qualities of water for reuse in the apartments, a treatment plant consisting of addition of chemicals, microstraining, pressure sand filtration, and carbon filtration will be added to the primary settling facilities in various combinations.

C. System Description

1. General

The objective of this plan is two fold:

(a) To demonstrate the collection of storm water on the selected area in storm retention ponds where it will be settled, disinfected, and discharged back to the stream, and

(b) To demonstrate the reuse possibilities of storm water by providing a plant capable of producing the various classes of water discussed for limited domestic use, including the distribution system and detailed plumbing to the test apartments.

The selected area was divided into three sub-areas each having its own retention pond as shown on Figure F-1. Below the third pond is a sampling and gaging station which will measure the actual flow released and take periodic samples of the water.

Sampling and gaging station No. 2, shown in the area north of the selected area, will provide flow and water quality data from an untreated area for comparison purposes.

All flow is by gravity so that no interceptors or pumping stations are required.

2. Flows

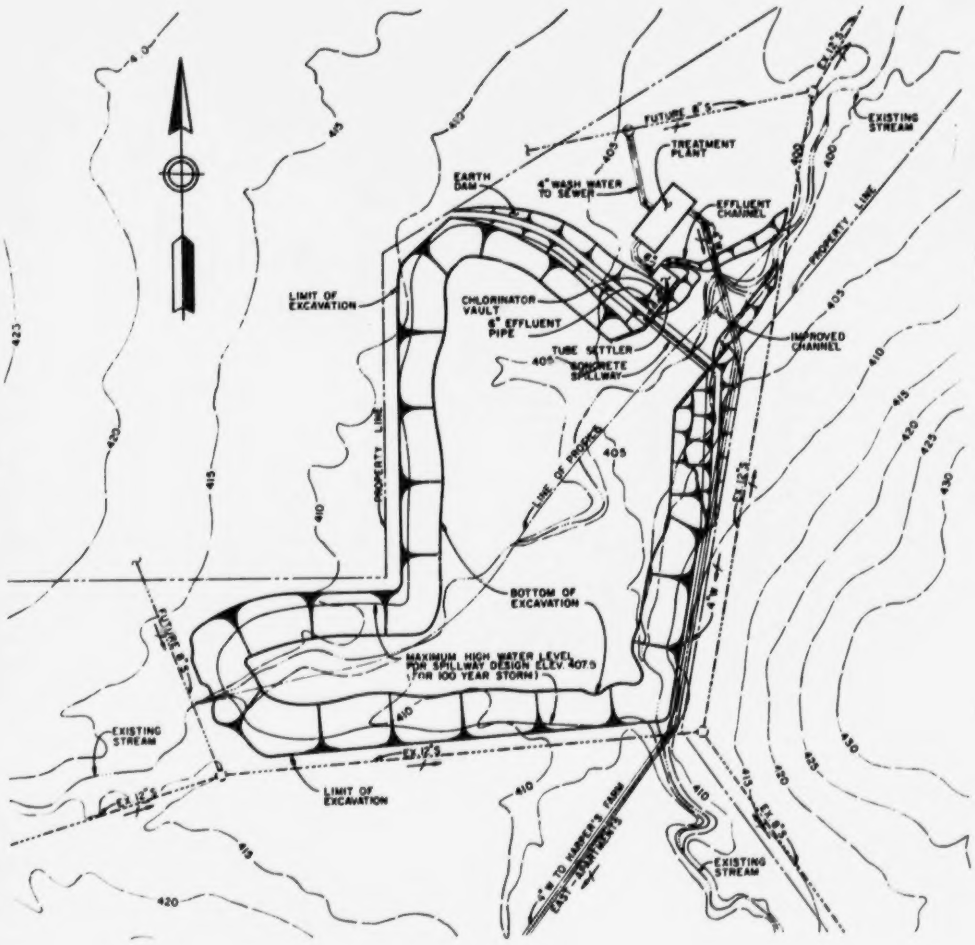
Flows have been computed utilizing the one year storm curve from Figure D-1, Appendix D, for the various areas tributary to the retention ponds and utilizing the rational method to compute the peak flows for the sampling and gaging stations. The volumes and flows to be expected from a one year storm on these areas are shown in Table F-1.

TABLE F-1. TRIBUTARY AREAS, STORM VOLUMES AND FLOWS

Area Description	Area (Acres)	One Year Storm	
		Total Volume (MG)	Peak Flow (cfs)
1. Tributary to storm retention pond No. 1	104	1.7	-
2. Tributary to storm retention pond No. 2	75.3	1.05	-
3. Tributary to storm retention pond No. 3 (not including 1 and 2)	75.1	1.28	-
4. Total tributary to sampling and gaging station No. 1 (including 1, 2, and 3)	285	-	131
5. Total tributary to sampling and gaging station No. 2	330	-	138

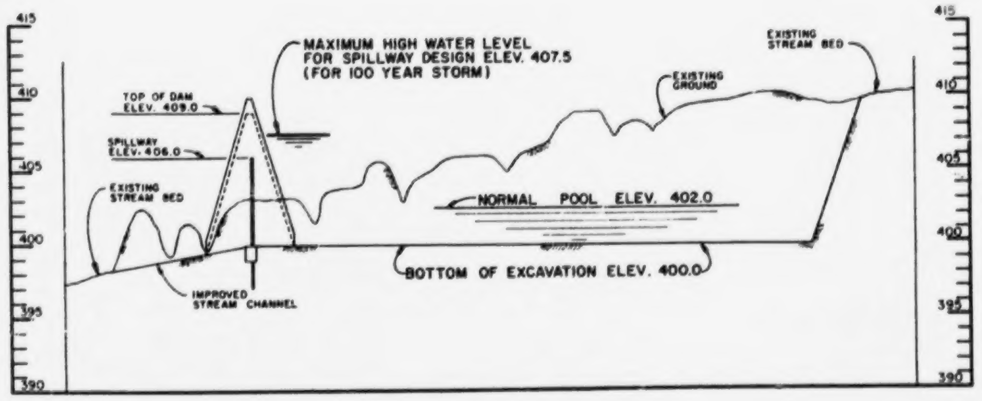
3. Storm Retention Ponds

The three storm retention ponds are located in available open space within the selected area. (See Figure F-1). The ponds, shown in detail in Figure F-2 and Figure F-3, consist of earth dams and excavated basins capable of providing storage for the one year design storm plus some silt storage.


EXISTING STREAM
TREATMENT PLANT
FUTURE 8"S.
EARTH DAM
4" WASH WATER TO SEWER
EFFLUENT CHANNEL
LIMIT OF EXCAVATION
CHLORINATOR VAULT
6" EFFLUENT PIPE
TUBE SETTLER
CONCRETE SPILLWAY
IMPROVED CHANNEL
PROPERTY LINE
LINE OF PROFILE
BOTTOM OF EXCAVATION
MAXIMUM HIGH WATER LEVEL FOR SPILLWAY DESIGN ELEV. 407.5 (FOR 100 YEAR STORM)
EX. 12"S
EX. 8"S
4"W
4"W TO HARPER'S FARM EAST APARTMENTS
410
415
420
423
425
430
405
400

SITE PLAN

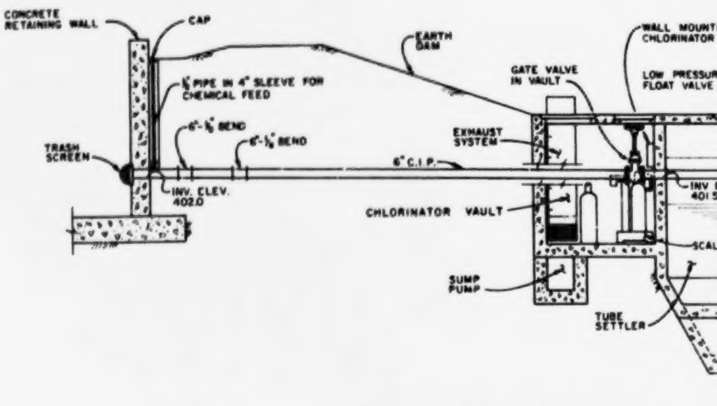
SCALE: 1" = 50'


415
410
405
400
395
390
MAXIMUM HIGH WATER LEVEL FOR SPILLWAY DESIGN ELEV. 407.5 (FOR 100 YEAR STORM)
EXISTING STREAM BED
EXISTING GROUND
TOP OF DAM ELEV. 409.0
SPILLWAY ELEV. 406.0
EXISTING STREAM BED
NORMAL POOL ELEV. 402.0
BOTTOM OF EXCAVATION ELEV. 400.0
IMPROVED STREAM CHANNEL

PROFILE

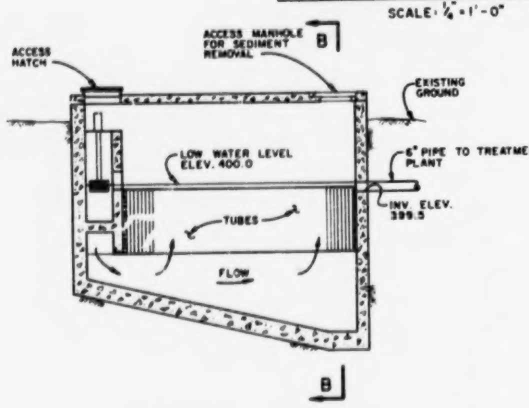
SCALE: HORIZ. 1" = 50'
VERT. 1" = 5'

STORM RETENTION POND NO. 1


CONCRETE RETAINING WALL
CAP
EARTH DAM
WALL MOUNT
CHLORINATOR
LOW PRESSUR
FLOAT VALVE
GATE VALVE IN VAULT
1/2" PIPE IN 4" SLEEVE FOR CHEMICAL FEED
6"-1/8" BEND
6"-1/8" BEND
EXHAUST SYSTEM
TRASH SCREEN
6" C.I.P.
INV. ELEV. 402.0
CHLORINATOR VAULT
INV
401.5
SCAL
SUMP PUMP
TUBE SETTLER

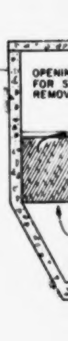
EFFLUENT PROFILE

SCALE: 1/4" = 1'-0"


ACCESS HATCH
ACCESS MANHOLE FOR SEDIMENT REMOVAL
B
EXISTING GROUND
LOW WATER LEVEL ELEV. 400.0
6" PIPE TO TREATMENT PLANT
INV. ELEV. 399.5
TUBES
FLOW
B

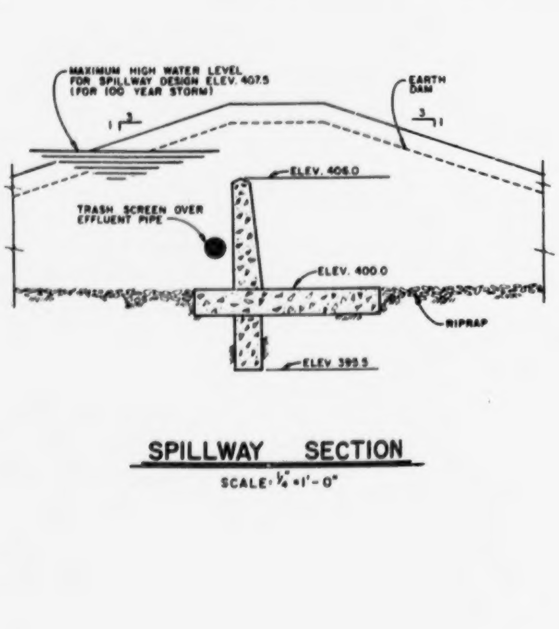
SECTION A-A

SCALE: 1/4" = 1'-0"


OPENI
FOR S
REMOV

SECT
SCA

TUBE SETTLER DETA


MAXIMUM HIGH WATER LEVEL FOR SPILLWAY DESIGN ELEV. 407.5 (FOR 100 YEAR STORM)
EARTH DAM
1 3
3 1
ELEV. 406.0
TRASH SCREEN OVER EFFLUENT PIPE
ELEV. 400.0
RIPRAP
ELEV. 399.5

SPILLWAY SECTION

SCALE: 1/4" = 1'-0"

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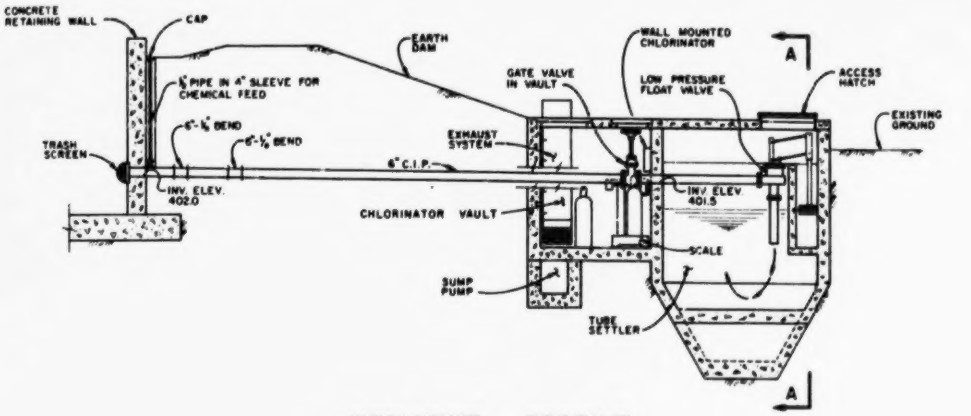
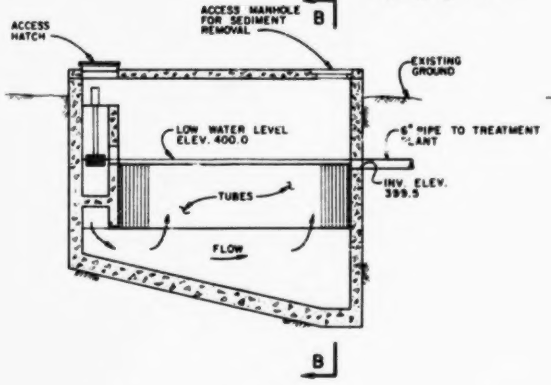
W.O. 5938

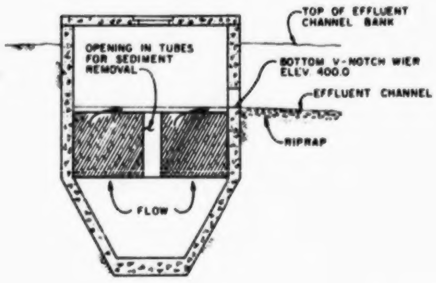
A


CONCRETE RETAINING WALL
CAP
EARTH DAM
WALL MOUNTED CHLORINATOR
A
ACCESS HATCH
½" PIPE IN 4" SLEEVE FOR CHEMICAL FEED
GATE VALVE IN VAULT
LOW PRESSURE FLOAT VALVE
EXISTING GROUND
6"-⅛" BEND
6"-⅛" BEND
EXHAUST SYSTEM
TRASH SCREEN
6" C.I.P.
INV. ELEV. 402.0
INV. ELEV. 401.5
CHLORINATOR VAULT
SCALE
SUMP PUMP
TUBE SETTLER
A

EFFLUENT PROFILE

SCALE: ¼" = 1'-0"


ACCESS HATCH
ACCESS MANHOLE FOR SEDIMENT REMOVAL
B
EXISTING GROUND
LOW WATER LEVEL ELEV. 400.0
6" PIPE TO TREATMENT PLANT
INV. ELEV. 399.5
TUBES
FLOW
B


TOP OF EFFLUENT CHANNEL BANK
OPENING IN TUBES FOR SEDIMENT REMOVAL
BOTTOM V-NOTCH WIER ELEV. 400.0
EFFLUENT CHANNEL
RIPRAP
FLOW

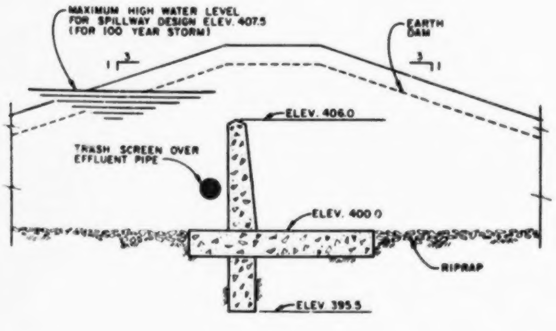
SECTION A-A

SCALE: ¼" = 1'-0"

SECTION B-B

SCALE: ¼" = 1'-0"

TUBE SETTLER DETAILS


MAXIMUM HIGH WATER LEVEL FOR SPILLWAY DESIGN ELEV. 407.5 (FOR 100 YEAR STORM)
EARTH DAM
1
3
3
1
ELEV. 406.0
TRASH SCREEN OVER EFFLUENT PIPE
ELEV. 400.0
RIPRAP
ELEV. 395.5

SPILLWAY SECTION

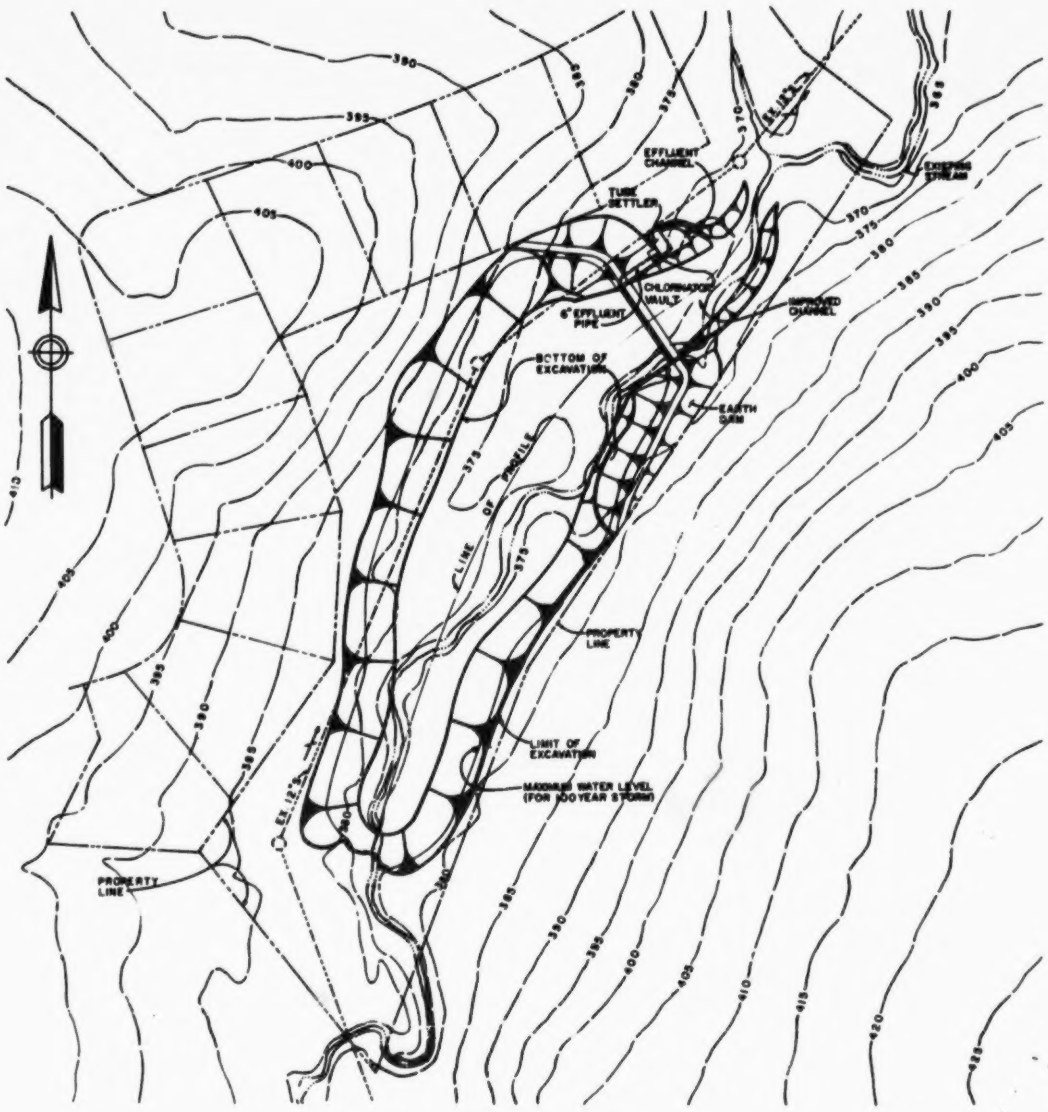
SCALE: ¼" = 1'-0"

NOTE: DETAILS OF THE SETTLER, EFFLUENT PIPING AND SPILLWAY ARE SHOWN FOR STORM RETENTION POND NO. 1 DETAILS FOR POND NO. 2 AND POND NO. 3 ARE SIMILAR.

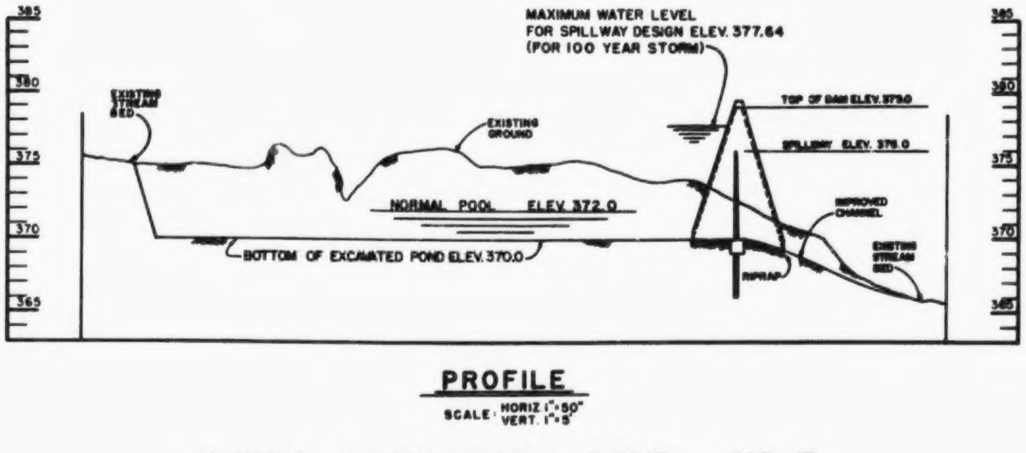
STORM RETENTION POND NO. 1
AND
TUBE SETTLER DETAILS

B

Figure F-2

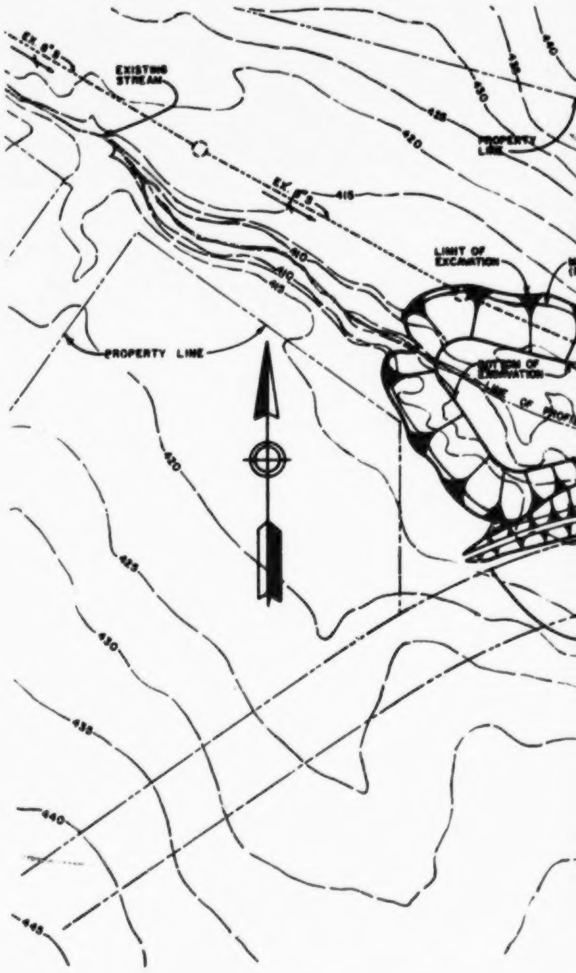

EFFLUENT CHANNEL
TUBE SETTLER
CHLORINATOR VAULT
6" EFFLUENT PIPE
IMPROVED CHANNEL
EXISTING STREAM
BOTTOM OF EXCAVATION
EARTH DAM
LINE OF PROFILE
PROPERTY LINE
LIMIT OF EXCAVATION
MAXIMUM WATER LEVEL (FOR 100 YEAR STORM)
EX. 12"S
PROPERTY LINE
EX. 12"S
365
370
375
380
385
390
395
400
405
410
413
415
420
425

SITE PLAN
SCALE: 1"=50'


MAXIMUM WATER LEVEL FOR SPILLWAY DESIGN ELEV. 377.64 (FOR 100 YEAR STORM)
EXISTING STREAM BED
EXISTING GROUND
TOP OF DAM ELEV. 379.0
SPILLWAY ELEV. 376.0
NORMAL POOL ELEV. 372.0
BOTTOM OF EXCAVATED POND ELEV. 370.0
IMPROVED CHANNEL
EXISTING STREAM BED
RIPRAP
385
380
375
370
365

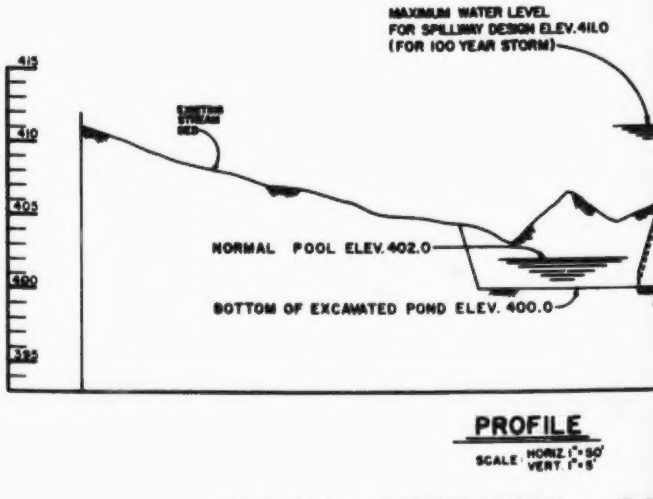
PROFILE
SCALE: HORIZ. 1"=50' VERT. 1"=5'

STORM RETENTION POND NO. 3


EX. 6"S
EXISTING STREAM
PROPERTY LINE
EX. 6"S
415
LIMIT OF EXCAVATION
PROPERTY LINE
BOTTOM OF EXCAVATION
LINE OF PROFILE
410
415
420
425
430
435
440
445

SITE PLA
SCALE: 1"=50'

NOTE: DETAILS OF
AND TUBE SET
IN FIGURE 2


MAXIMUM WATER LEVEL FOR SPILLWAY DESIGN ELEV. 411.0 (FOR 100 YEAR STORM)
EXISTING STREAM BED
NORMAL POOL ELEV. 402.0
BOTTOM OF EXCAVATED POND ELEV. 400.0
415
410
405
400
395

PROFILE
SCALE: HORIZ. 1"=50' VERT. 1"=5'

STORM RETENTION PO

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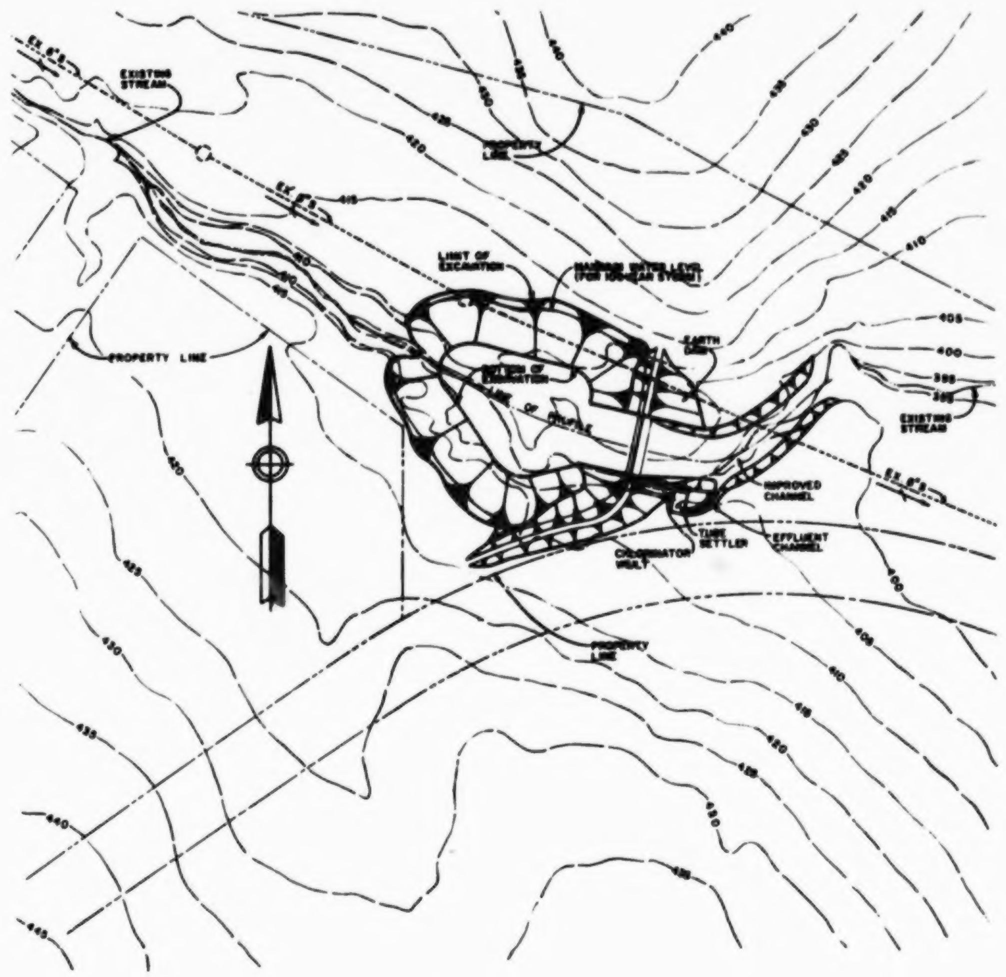
A

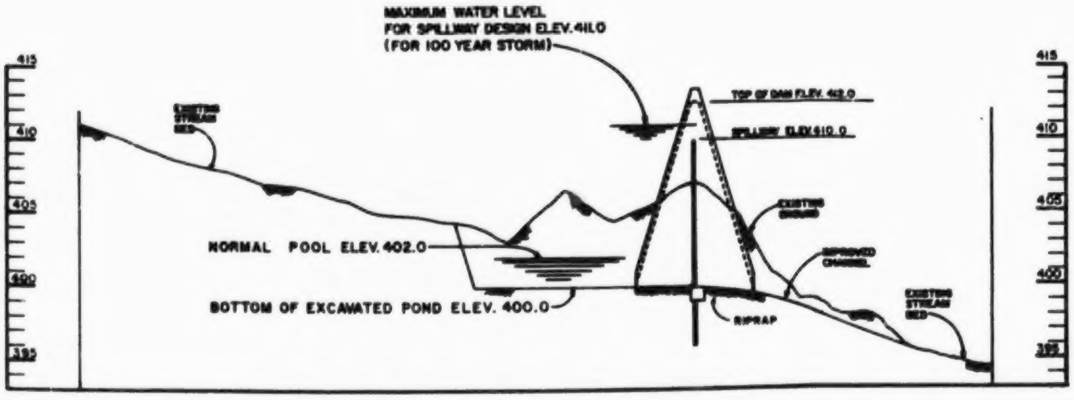
EXISTING STREAM
395
400
405
425
395
390
375
370
365


EX. 8"S.
EXISTING STREAM
PROPERTY LINE
440
435
430
425
420
415
410
LIMIT OF EXCAVATION
MAXIMUM WATER LEVEL (FOR 100-YEAR STORM)
EARTH DAM
BOTTOM OF EXCAVATION
LINE OF PROFILE
405
400
395
390
EXISTING STREAM
IMPROVED CHANNEL
TUBE SETTLER
EFFLUENT CHANNEL
CHLORINATOR VAULT
PROPERTY LINE
PROPERTY LINE

SITE PLAN

SCALE: 1"=50'

NOTE: DETAILS OF CONCRETE SPILLWAYS, EFFLUENT PIPING AND TUBE SETTLERS ARE SIMILAR TO THOSE SHOWN IN FIGURE 2


MAXIMUM WATER LEVEL
FOR SPILLWAY DESIGN ELEV. 411.0
(FOR 100 YEAR STORM)
TOP OF DAM ELEV. 412.0
SPILLWAY ELEV. 410.0
EXISTING STREAM BED
EXISTING GROUND
NORMAL POOL ELEV. 402.0
IMPROVED CHANNEL
BOTTOM OF EXCAVATED POND ELEV. 400.0
RIPRAP
EXISTING STREAM BED
415
410
405
400
395

PROFILE

SCALE: HORIZ. 1"=50'
VERT. 1"=5'

STORM RETENTION POND NO. 2

STORM RETENTION PONDS
NO. 2 AND NO. 3

Figure F-3

B

The concrete spillway section is designed in each case to handle peak flow from a 100 year storm in accordance with the requirements of the State of Maryland Department of Water Resources', "Regulations and Rules of Procedure," regarding small dams, reservoirs, and waterway obstructions.

It should be noted that because of the overflow rate projected for the tube settlers, the probability of consecutive day storms overtopping the spillway is increased. Until the optimum overflow rate for the tube settlers is determined by actual demonstration, no detailed statistical analysis will be attempted for the frequency of spillway overflow.

4. Tube Settlers

The tube settlers, shown on Figure F-2 have been selected and designed to provide a mechanical means of improvement of the natural settlement provided by the retention pond. It is anticipated that the pond and settler operating together will provide the desired sedimentation results.

As previously mentioned, maximum rate of flow through the settlers remains to be demonstrated. It is judged, however, that this rate is in the vicinity of twice the annual mean runoff flow for the area being processed. These flow rates and the length of time required to process a one year storm are shown in Table F-2.

TABLE F-2. STORM VOLUMES AND PROCESSING TIMES

Area	1 Year Storm Volume (MG)	Twice Annual Average Flow (gpm)	Time Required to Process 1 Year Storm (days)
1	1.7 MG	108	10.9
2	1.05 MG	58	12.5
3	1.28 MG	80	11.1

It is anticipated that the tube settlers will demonstrate themselves capable of an increased overflow rate and, thus, decrease the time required to pass the one year storm volume and thereby reduce the volume of untreated spillway overflow due to consecutive storms.

It should be noted that storm retention pond No. 3 will reprocess the treated water from pond No. 2 and pond No. 1. This will require the overflow rate through the tube settler at this pond to be the sum of the rates indicated in Table F-2 (i.e., 246 gpm) to prevent overflow of the spillway.

This arrangement of the ponds, that is, two ponds in parallel feeding the third pond, will provide the maximum flexibility for demonstration of treatment possibilities.

5. Chemical Treatment

a. Flocculation. Provisions have been incorporated to allow chemical additions to the water just after leaving the pond and prior to reaching the tube settler (See Figure F-2) so that additional clarification of turbidity by chemical means in the tube settler may be attempted.

b. Disinfection. A wall-mounted chlorinator is provided in a concrete vault adjacent to the tube settlers as detailed on Figure F-2. The rate of chlorine feed will be automatically regulated by flow rate. The actual amount of chlorine required to meet the criteria of Class "C" water should be based on actual conditions and observed results. It may be assumed that a dosage of three to five ppm is realistic for an average condition of the storm water.

6. Sampling and Gaging Stations

The sampling and gaging stations shown in Figure F-4 consist of a combination V-notch and rectangular measuring weir, a water level recording device and an automatic water sampling device. The wier, located within an earth dam and concrete spillway, will measure up to 1.6 cfs within its V-notch and up to one year design storm within its rectangular portion. The concrete spillway is designed to pass a 100 year storm in accordance with State of Maryland Department of Water Resources regulations previously quoted.

Station No. 1 will record flow and collect samples of treated water from the system of storm retention ponds and tube settlers. Station No. 2 will record the flow and collect samples of untreated water from an area of similar size and characteristics.

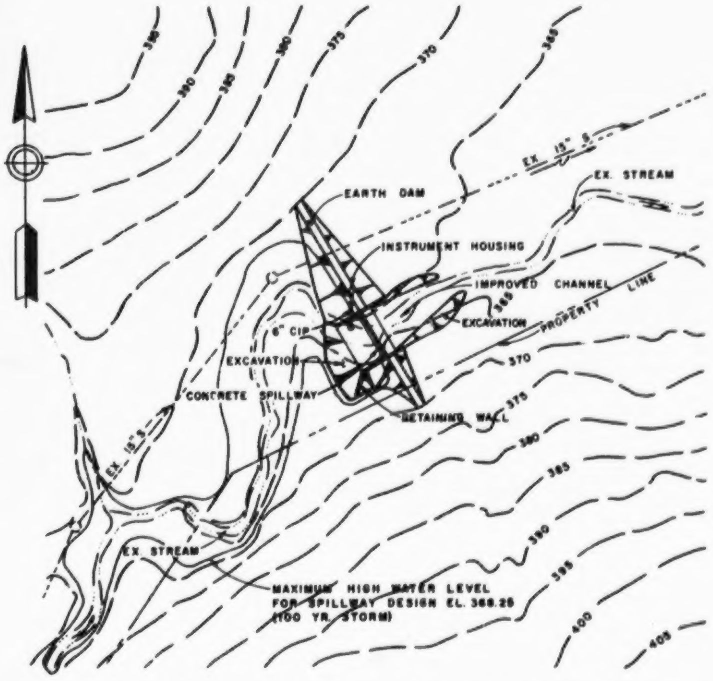
Comparison of results from the two stations will enable evaluation of the treatment provided and demonstrate the effect of the ponds on the reduction of peak storm flows and erosion.

7. Water Treatment Plant

The water treatment plan will have the ability to take the Class "C" water from the tube settlers and, through various treatment combinations, produce Class "B", Class "A", or Class "AA" water for reuse. Figure F-5 shows a conceptual plan and elevation of the treatment plant.

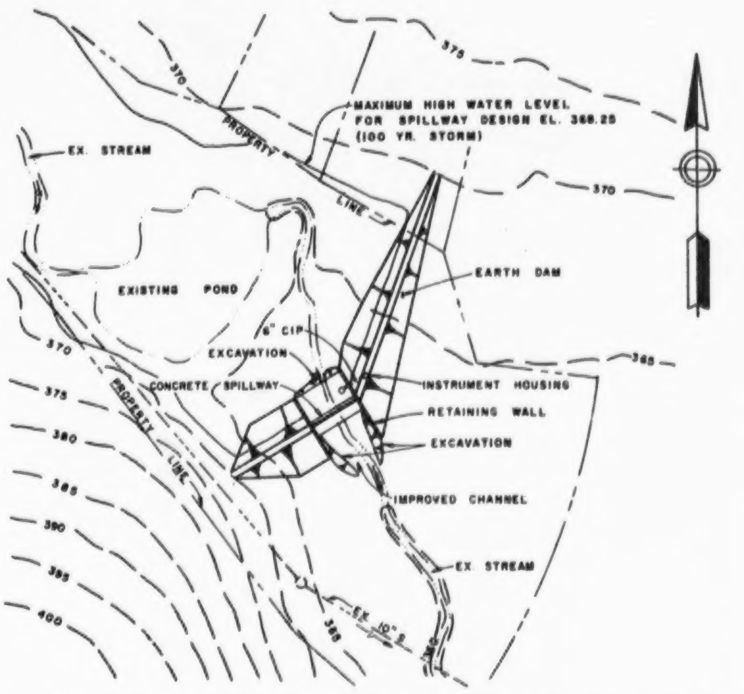
Water from the tube settler is delivered to the 1500 gallon wet well of the treatment plant through a six inch pipe. A float valve in the wet well controls the feed to the treatment plant whenever the reuse system has a demand put on it. During periods of low flow when the tube settler cannot meet the system demands, make up water from the local county system will be utilized.

The average demand anticipated from the apartment site connection is 15,000 gallons per day, with a 30,000 gallons per day peak flow. At this rate of flow through the plant the wet well has a retention time of approximately one hour which is ample contact time for chemical feeding and aeration purposes.


380
390
385
380
375
370
365
EX. 15" S
EX. STREAM
EARTH DAM
INSTRUMENT HOUSING
IMPROVED CHANNEL
PROPERTY LINE
365
EXCAVATION
6" CIP
EXCAVATION
370
CONCRETE SPILLWAY
RETAINING WALL
375
EX. 15" S
380
385
EX. STREAM
MAXIMUM HIGH WATER LEVEL
FOR SPILLWAY DESIGN EL. 368.25
(100 YR. STORM)
390
395
400
405

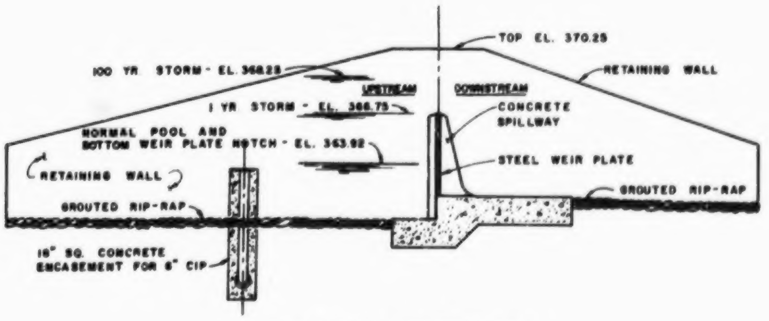
SITE PLAN - SAMPLING AND GAGING STATION NO. 1

SCALE 1" = 50'


370
375
MAXIMUM HIGH WATER LEVEL
FOR SPILLWAY DESIGN EL. 368.25
(100 YR. STORM)
PROPERTY LINE
EX. STREAM
370
EXISTING POND
EARTH DAM
6" CIP
EXCAVATION
365
370
CONCRETE SPILLWAY
INSTRUMENT HOUSING
RETAINING WALL
375
EXCAVATION
PROPERTY LINE
380
385
IMPROVED CHANNEL
390
EX. STREAM
395
400
EX. 10" S
365

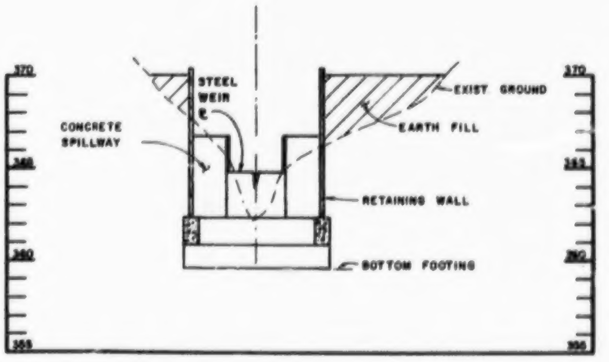
SITE PLAN - SAMPLING AND GAGING STATION NO. 2

SCALE 1" = 50'


TOP EL. 370.25
100 YR. STORM - EL. 368.25
RETAINING WALL
UPSTREAM
DOWNSTREAM
1 YR. STORM - EL. 366.75
CONCRETE SPILLWAY
NORMAL POOL AND
BOTTOM WEIR PLATE NOTCH - EL. 363.92
STEEL WEIR PLATE
RETAINING WALL
GROUTED RIP-RAP
GROUTED RIP-RAP
16" SQ. CONCRETE
ENCASEMENT FOR 6" CIP

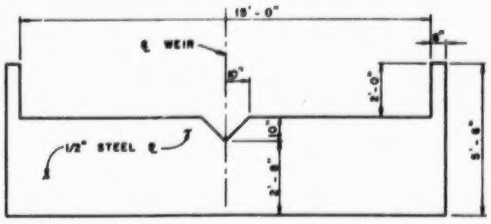
SPILLWAY SECTION

SCALE 1/4" = 1'-0"


370
370
STEEL
WEIR
℄
EXIST. GROUND
CONCRETE
SPILLWAY
EARTH FILL
365
365
RETAINING WALL
360
360
BOTTOM FOOTING
355
355

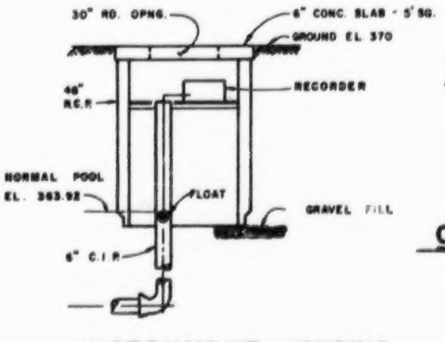
FRONT ELEVATION OF DAM

SCALE HORZ. 1" = 20'
VERT. 1" = 4'


15'-0"
6"
℄ WEIR
10"
2'-0"
1/2" STEEL ℄
10"
3'-6"
2'-8"

WEIR PLATE DETAIL

SCALE 3/8" = 1'-0"


30" RD. OPNG.
6" CONC. SLAB - 5' SQ.
GROUND EL. 370
48" R.C.P.
RECORDER
NORMAL POOL
EL. 363.92
FLOAT
GRAVEL FILL
6" C.I.P.

INSTRUMENT HOUSING DETAIL

SCALE 3/8" = 1'-0"

WEIR PLATE
ANCHOR BOLT

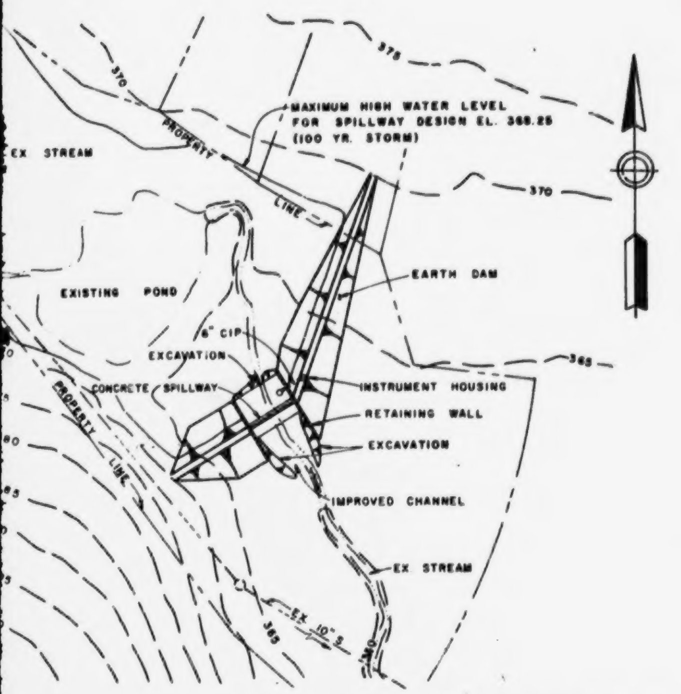
WEIR PLATE CONNECTION DETAIL

SCALE 1" = 1'-0"

WHITMAN, REQUARDT & ASSOCIATES
ENGINEERS
BALTIMORE MARYLAND

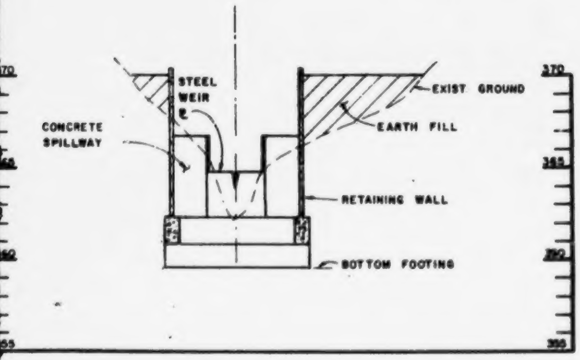
W.O. 5938

A


375
370
MAXIMUM HIGH WATER LEVEL FOR SPILLWAY DESIGN EL. 368.25 (100 YR. STORM)
EX STREAM
PROPERTY LINE
370
EXISTING POND
EARTH DAM
6" CIP
EXCAVATION
365
CONCRETE SPILLWAY
INSTRUMENT HOUSING
PROPERTY LINE
RETAINING WALL
EXCAVATION
IMPROVED CHANNEL
EX STREAM
EX 10" S
365
360

PLAN- SAMPLING AND GAGING STATION NO. 2

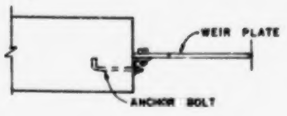
SCALE 1"=50'


370
STEEL WEIR ℄
EXIST GROUND
CONCRETE SPILLWAY
EARTH FILL
365
RETAINING WALL
BOTTOM FOOTING
360
355

FRONT ELEVATION OF DAM

SCALE HORZ 1"=20' VERT 1"=4'

SLAB - 5' SQ.
370
FILL


WEIR PLATE
ANCHOR BOLT

WEIR PLATE CONNECTION DETAIL

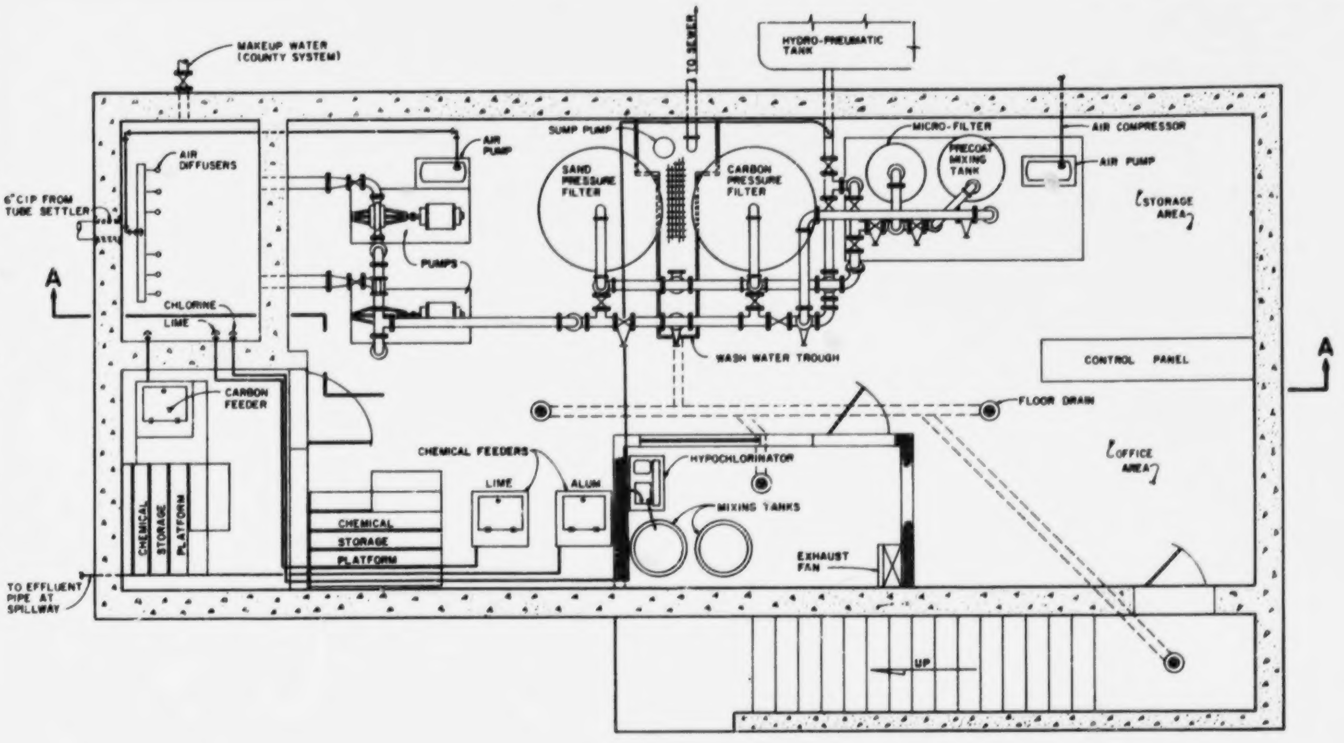
SCALE 1"=1'-0"

NG

SAMPLING AND GAGING STATIONS NO. 1 AND NO. 2

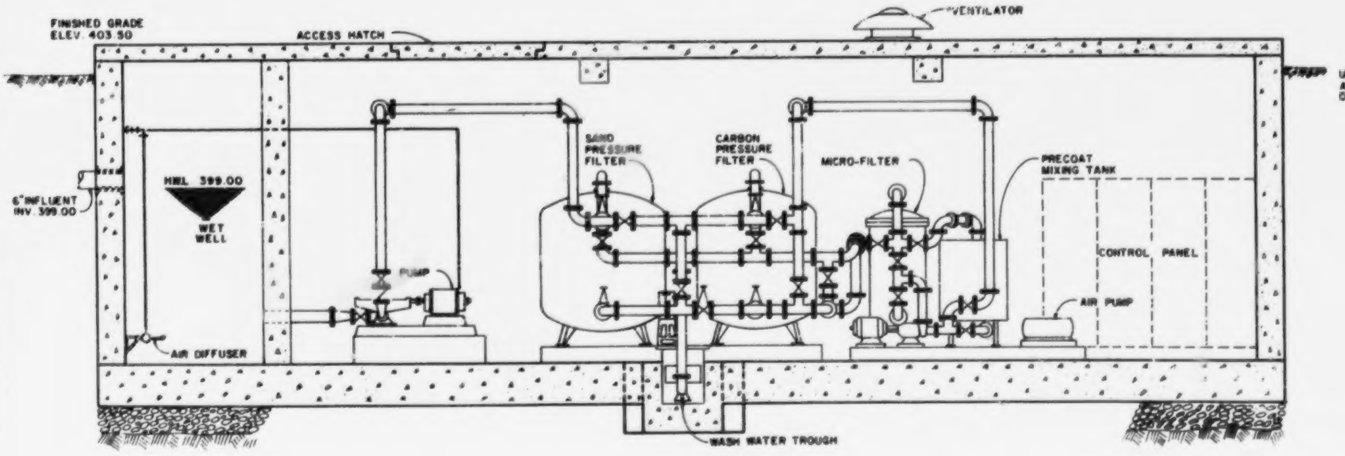
Figure F-4

B


TO SEWER
HYDRO-PNEUMATIC TANK
MAKEUP WATER (COUNTY SYSTEM)
SUMP PUMP
MICRO-FILTER
AIR COMPRESSOR
AIR PUMP
AIR DIFFUSERS
SAND PRESSURE FILTER
CARBON PRESSURE FILTER
PRECOAT MIXING TANK
AIR PUMP
STORAGE AREA
6" CIP FROM TUBE SETTLER
PUMPS
A
A
CHLORINE
LIME
WASH WATER TROUGH
CONTROL PANEL
CARBON FEEDER
FLOOR DRAIN
CHEMICAL FEEDERS
HYPOCHLORINATOR
LIME
ALUM
OFFICE AREA
MIXING TANKS
CHEMICAL STORAGE PLATFORM
CHEMICAL
STORAGE
PLATFORM
EXHAUST FAN
TO EFFLUENT PIPE AT SPILLWAY
UP

PLAN

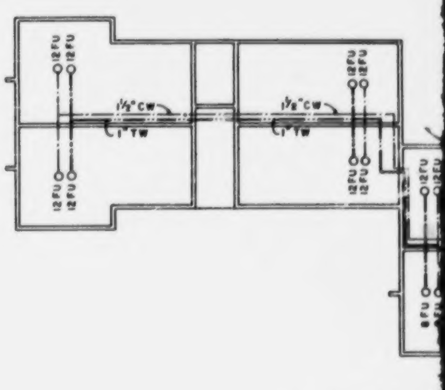
SCALE : 3/8" = 1'-0"


FINISHED GRADE ELEV. 403.50
ACCESS HATCH
VENTILATOR
6" INFLUENT INV. 399.00
HWL 399.00
WET WELL
SAND PRESSURE FILTER
CARBON PRESSURE FILTER
MICRO-FILTER
PRECOAT MIXING TANK
PUMP
CONTROL PANEL
AIR PUMP
AIR DIFFUSER
WASH WATER TROUGH

SECTION A-A

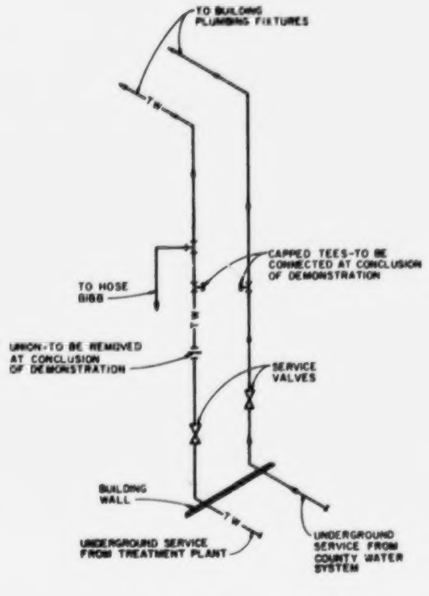
SCALE: 3/8" = 1'-0"

WATER TREATMENT PLANT


12FU
12FU
1½" CW
1" TW
1½" CW
1" TW
12FU
12FU
12FU
12FU
12FU
12FU
8FU

GROUND LEV

TYPICAL WATE


TO BUILDING PLUMBING FIXTURES
TW
CAPPED TEES-TO BE CONNECTED AT CONCLUSION OF DEMONSTRATION
TO HOSE BIBB
UNION-TO BE REMOVED AT CONCLUSION OF DEMONSTRATION
SERVICE VALVES
BUILDING WALL
TW
UNDERGROUND SERVICE FROM TREATMENT PLANT
UNDERGROUND SERVICE FROM COUNTY WATER SYSTEM

DETAIL "A"

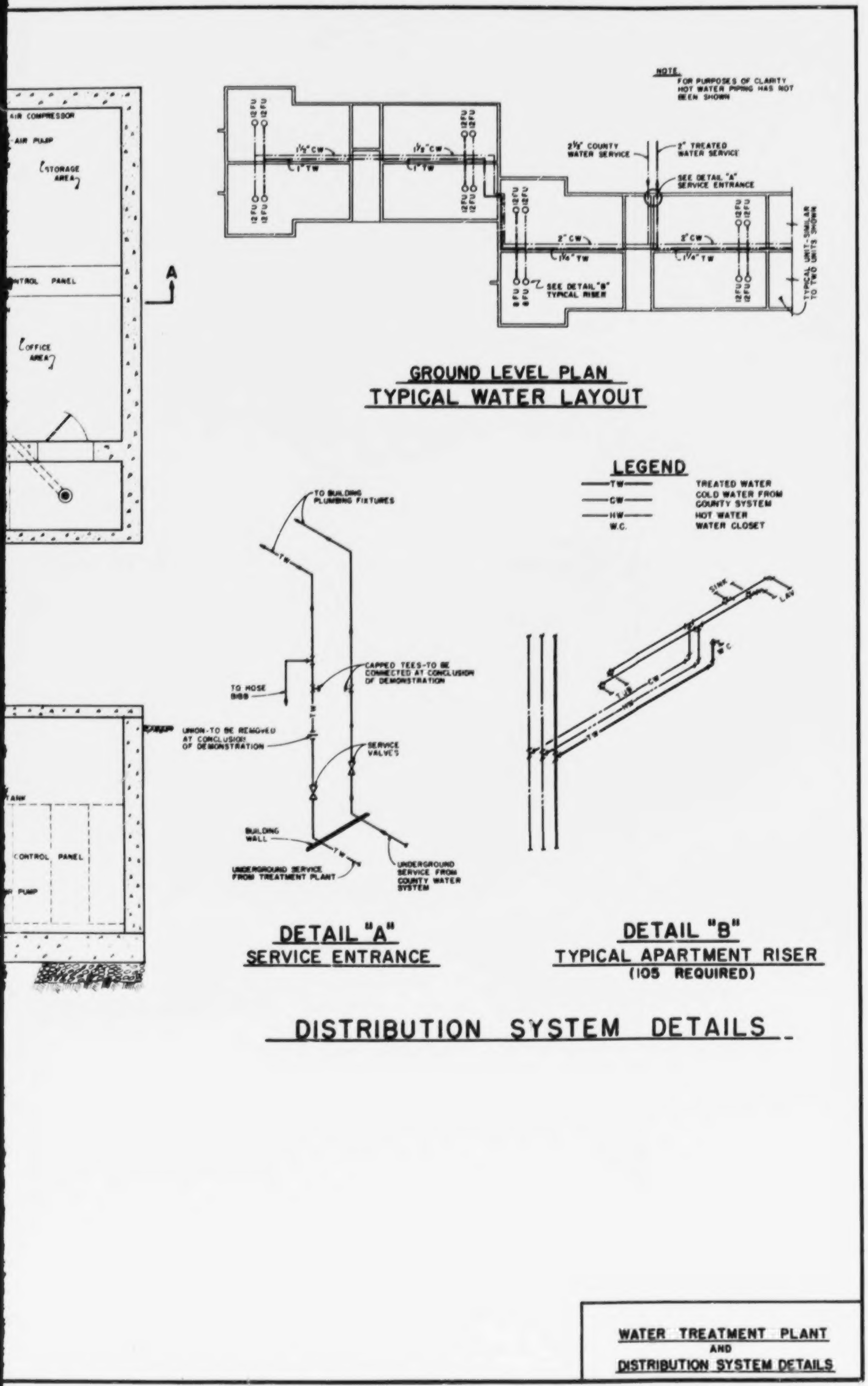
SERVICE ENTRANCE

DISTRIBUTION S

WHITMAN REQUARDT & ASSOCIATES
ENGINEERS
BALTIMORE MARYLAND

W.O. 5938

A


AIR COMPRESSOR
AIR PUMP
STORAGE AREA
NTROL PANEL
A
OFFICE AREA
TANK
CONTROL PANEL
R PUMP
NOTE
FOR PURPOSES OF CLARITY HOT WATER PIPING HAS NOT BEEN SHOWN
12FU
1½" CW
1" TW
2½" COUNTY WATER SERVICE
2" TREATED WATER SERVICE
SEE DETAIL "A" SERVICE ENTRANCE
2" CW
1¼" TW
8FU
SEE DETAIL "B" TYPICAL RISER
TYPICAL UNIT-SIMILAR TO TWO UNITS SHOWN
GROUND LEVEL PLAN
TYPICAL WATER LAYOUT
LEGEND
TW TREATED WATER
CW COLD WATER FROM COUNTY SYSTEM
HW HOT WATER
W.C. WATER CLOSET
TO BUILDING PLUMBING FIXTURES
TW
TO HOSE BIBB
CAPPED TEES-TO BE CONNECTED AT CONCLUSION OF DEMONSTRATION
UNION-TO BE REMOVED AT CONCLUSION OF DEMONSTRATION
SERVICE VALVES
BUILDING WALL
UNDERGROUND SERVICE FROM TREATMENT PLANT
UNDERGROUND SERVICE FROM COUNTY WATER SYSTEM
SINK
LAV
W.C
CW
HW
TW
DETAIL "A"
SERVICE ENTRANCE
DETAIL "B"
TYPICAL APARTMENT RISER
(105 REQUIRED)
DISTRIBUTION SYSTEM DETAILS
WATER TREATMENT PLANT AND DISTRIBUTION SYSTEM DETAILS

B

Figure F-5

The following operations will be accomplished in the wet well if required:

(a) Chemical addition for PH control

(b) Chlorination for taste and odor control, and for disinfection

(c) Powdered carbon for coating the sand filters, for taste and odor control, and/or for removal of organic material

(d) Aeration

One 20 gpm pump will provide the design flow and required head through the treatment plant to the hydro-pneumatic tank and into the distribution system. A second 20 gpm pump will provide back-up for emergencies and additional capacity if required.

From the wet well through the pumps the water may be passed through a Micro-Filter similar to Croll Reynolds Engineering Company, Inc., Precoat Filter, a pressure sand filter and/or a carbon filter either individually or in any combination. A further refinement may be achieved by precoating the Micro-Filter with a diatomaceous earth or a clay slurry.

The actual degree of treatment required and the equipment to be utilized to achieve the various classes of water depends primarily on the quality of the water coming into the wet well. The plant will have complete flexibility to provide the various treatments in whatever combinations are necessary. It is anticipated that the following treatment will be required to produce the various classes of water listed:

(a) Class "B"

    (1) Ph control

    (2) Micro Filtration

    (3) Disinfection (If necessary)

(b) Class "A"

    (1) Aeration

    (2) Ph control

    (3) Softening (If necessary)

    (4) Disinfection

    (5) Pressure sand filtration

(6) Carbon filtration (If color and odor problems require)

(c) Class "AA"

(1) Aeration

(2) Ph control

(3) Softening (If necessary)

(4) Disinfection

(5) Pressure sand filtration

(6) Carbon filtration (If necessary)

(7) Micro-Filtration with precoated filters (If necessary)

Following treatment, the water flows to a 7,000 gallon hydro-pneumatic tank where it will await demand from the apartment site. The tank has been sized to provide backwash quantities to the filters at the required pressures. The plant will be provided with sampling and testing equipment to analyze the various treatment procedures as well as to control the treatment required.

8. Water Distribution System

The distribution system from the treatment plant to the Harper's Farm East Apartments consists of approximately 2000 feet of four inch and two inch water main as shown on Figure F-1. The entrance to the three groups of apartments will parallel the normal service connections from the county water main.

A water distribution system will be installed within each of the three selected groups of apartments as detailed schematically in the plan and isometric diagrams on Figure F-5. The detailed installation of the system will be in accordance with the plumbing regulations of the State of Maryland Health Department and with the Howard County Building Code particularly with regard to cross connections. This system will serve the water closets for the 105 apartments and the exterior hose connections, and will be placed parallel to the normal cold water system.

The system will be constructed so that at the completion of the demonstration program normal service may be restored by one simple connection between the two cold water pipelines and the removal of one section of pipe as shown on Figure F-5. The demonstration piping would then be abandoned.

D. Costs

Construction costs for the demonstration program are based on early 1968 construction unit prices developed by updating recorded construction prices for similar types of work. These prices represent an estimate of the total contractor's bid including his direct costs, overhead, and profit.

To arrive at the project costs it was necessary to increase the construction costs by the following percentages to reflect additional charges experienced in this type of construction:

Construction Contingencies	15%
Project Cost (Engineering, Legal, Administrative, etc)	20%
Escalation (per year)	5%
TOTAL INCREASE TO BASIC CONSTRUCTION COST	40%

CONSTRUCTION COSTS:

1. Storm Retention Pond No. 1:

Site Work	$12,500.	
Excavation	31,000.	
Embankment	1,200.	
Concrete Spillway and Walls	11,500.	
Grouted Rip Rap	5,000.	
Effluent Piping, Chlorinator and Tube Settler	9,200.	
Total - Pond No. 1	$70,400.	$ 70,400.

2. Storm Retention Pond No. 2:

Site Work	$ 8,000.	
Excavation	15,000.	
Embankment	900.	
Concrete Spillway and Walls	17,500.	
Grouted Rip Rap	4,000.	
Effluent Piping, Chlorinator and Tube Settler	8,200.	
Total - Pond No. 2	$53,600.	$ 53,600.
SUBTOTAL - Items on this page		$124,000.

SUBTOTAL - Brought forward from page F-19			$124,000.
3.	Storm Retention Pond No. 3:		
	Site Work	$11,000.	
	Excavation	15,000.	
	Embankment	1,000.	
	Concrete Spillway and Walls	15,000.	
	Grouted Rip Rap	3,300.	
	Effluent Piping, Chlorinator and Tube Settler	10,200.	
	Total - Pond No. 3	$55,500.	$ 55,500.
4.	Sampling and Gaging Station No. 1:		
	Site Work	$ 6,000.	
	Excavation	250.	
	Embankment	750.	
	Concrete Spillway and Walls	4,500.	
	Grouted Rip Rap	1,000.	
	Weir Plate and Instrumentation	2,300.	
	Total - Station No. 1	$14,800.	$ 14,800.
5.	Sampling and Gaging Station No. 2:		
	Site Work	$ 6,300.	
	Excavation	150.	
	Embankment	1,500.	
	Concrete Spillway and Walls	5,500.	
	Grouted Rip Rap	1,700.	
	Weir Plate and Instrumentation	2,350.	
	Total - Station No. 2	$17,500.	$ 17,500.
6.	Treatment Plant		
	Excavation	$ 4,000.	
	Structure	20,000.	
	Mechanical	51,000.	
	Electrical	45,000.	
	Total - Treatment Plant	$120,000.	$120,000.
SUBTOTAL - Pages F-19 and F-20			$331,800.

SUBTOTAL - Brought forward from page F-20		$331,800.
7. Distribution System:		
4" and 2" Main to Apartment Site	$ 11,000.	
Interior Plumbing	15,000.	
Total - Distribution System	$ 26,000.	$ 26,000.
TOTAL CONSTRUCTION COST		$357,800.
Add 40% for Construction Contingencies, Project Costs, and Escalation for one Year		$143,200.
TOTAL PROJECT COST		$501,000.

APPENDIX G

DEMONSTRATION PROGRAM - OPERATION AND MAINTENANCE AND TEST AND EVALUATION

This appendix contains listings of the items of work that would be required in connection with the "Operation and Maintenance" and "Test and Evaluation" portions of the Demonstration Program. The following tables are included in this appendix:

Table G-1	Demonstration Program - Operation and Maintenance Costs
Table G-2	Demonstration Program - Test and Evaluation Costs
Table G-3	Application Projects - Operation and Maintenance Costs
Tabel G-4	Application Projects - Test and Evaluation Costs

Each of these tables contain a listing of the tasks and subtasks and the work items within each of the subtasks. Tables G-1 and G-2 show the costs associated with each of the four demonstration contract programs described in Chapter VII. Tables G-3 and G-4 contain the costs associated with the demonstration grant projects under the "Combined Demonstration Program" approach. The tables are arranged as follows:

(1) First Column - Listing of Tasks, Subtasks, and Items of Work

(2) Subsequent Columns - Demonstration Program Approaches subdivided as follows:

  (a) Number of Units or Applicability - Number designations where appropriate and "x" where an item of work applies.

  (b) Performance Period - Number of months over which work will be performed. Where number is not listed for individual items, the number of months for the subtask applies.

  (c) Labor Costs - In thousands of dollars, based on typical labor rates for the type of work involved and duration of the work including overhead.

  (d) Materials Costs - In thousands of dollars, including travel supplies, services, computer rental, special equipment, and allowances for modification.

  (e) Total Costs - In thousands of dollars, the sum of labor and materials costs.

TABLE G-1. DEMONSTRATION PROGRAM - OPERATION AND MAINTENANCE COSTS

(Costs in Thousands)

	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components - Alternate Plan					Combined Demonstration Program									
																Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
1. Operation and Maintenance																									
a. Gaging and Sampling Stations	2	16									2	16													
(1) Checkout and Calibration of Equipment	X										X														
(2) Weekly replacement of recorder charts	X										X														
(3) Quarterly cleaning and general maintenance	X										X														
(4) Repairs as required	X										X														
Subtotal			6.1	3.8	9.9								6.1	3.8	9.9										
b. Storage Reservoirs and Pretreatment Units	1 2	15 8				1	10				1 2	15 8				1 2	24 12								
(1) Weekly check and replenishment of chemicals	X					X					X					X									
(2) Weekly replacement of recorder charts	X					X					X					X					X				
(3) Bi-weekly cleaning of tube settler basins	X					X					X					X					X				
(4) Monthly preventative maintenance	X					X					X					X									
(5) Quarterly cleaning of reservoir	X					X					X					X									
(6) Nuisance control and general maintenance	X					X					X					X									
Subtotal			12.5	7.6	20.1			7.6	3.6	11.2			12.5	7.6	20.1		15.4	7.7	23.1						
c. Reuse Treatment Plant	1	8																			1	8			
(1) Checkout and initial operation	X																				X				
(2) Daily equipment checks, sample collection and chemical replenishment	X																				X				
(3) Daily collection of samples	X																				X				
(4) Monthly preventative maintenance	X																				X				
(5) Connection to Reuse System	X																				X				
(6) Repairs as required	X																				X				
(7) Electrical Energy	X																				X				
Subtotal			17.0	1.8	18.8			14.7	2.0	16.7													17.0	1.8	19.8
d. Packaged Treatment Plants and Pumping Facilities						1	10				1	7													
(1) Daily equipment checks, sample collection and chemical replenishment						X					X														
(2) Daily collection of samples						X					X														
(3) Monthly preventative maintenance						X					X														
(4) Electrical Energy						X					X														
Subtotal								14.7	2.0	16.7			12.6	1.4	14.0										
e. Reuse System	1	4				1	6				1	3									1	4			
(1) Initial connection	X					X					X										X				
(2) Installation of meters	X					X					X										X				
(3) Maintenance as required	X					X					X										X				
Subtotal			3.5	1.9	5.4			3.5	1.9	5.4			1.0	.5	1.5								1.5	.5	2.0
f. Operation and Maintenance Subtotal			39.1	15.1	54.2			25.8	7.5	33.3			32.2	13.3	45.5			15.4	7.7	23.1			18.5	2.3	21.8

TABLE G-2. DEMONSTRATION PROGRAM - TEST AND EVALUATION COSTS
(Costs in Thousands)

	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components Alternate Plan					Combined Demonstration Program									
																Pollution Control Contract					Storm Water Reuse Contract				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
2. Test and Evaluation																									
a. Program Management		24					21					24					30					24			
(1) Technical direction and coordination of overall program	X					X					X					X					X				
(2) Develop design criteria and guidelines	X					X					X					X					X				
(3) Select design subcontractor	X					X					X					X					X				
(4) Review and approval of drawings and specifications	X					X					X					X					X				
(5) Preparation of contract documents	X					X					X					X					X				
(6) Request bids	X					X					X					X					X				
(7) Evaluate bids and negotiate subcontracts	X					X					X					X					X				
(8) Monitor construction and field inspection	X					X					X					X					X				
(9) Approval field changes	X					X					X					X					X				
(10) Accept facilities after construction	X					X					X					X					X				
(11) Coordination with local and state agencies	X					X					X					X					X				
(12) Liasion with FWPCA	X					X					X					X					X				
(13) Approve test and evaluation program plan	X					X					X					X					X				
(14) Review test results	X					X					X					X					X				
(15) Direction of report preparation	X					X					X					X					X				
Subtotal			69.8	8.9	78.7			54.3	.5	54.8			69.8	.9	70.7			85.8	.9	86.7			72.0	8.5	80.5
b. Test Program Planning and Coordination		12					11					12					12					12			
(1) Prepare overall test program and schedule	X					X					X					X					X				
(2) Prepare forms for the compilation of construction costs	X					X					X					X					X				
(3) Prepare forms for the collection of operating and maintenance data	X					X					X					X					X				
(4) Determine special instrumentation requirements	X					X					X					X					X				
(5) Procure and install instrumentation and test equipment	X					X					X					X					X				
(6) Coordinate test scheduling	X					X					X					X					X				
(7) Plan and perform plant modifications	X	21				X	18				X	21				X	27				X	21			
Subtotal			21.8	34.0	55.8			15.8	11.6	27.4			21.8	34.0	55.8			26.3	21.2	47.5			26.2	19.5	45.7
c. Testing and Data Collection																									
(1) Design and Construction Data																									
(a) Analysis of bids and compilation of cost data	X	9				X	6				X	9				X	12				X	12			
(b) Review of field changes	X	9				X	6				X	9				X	12				X	12			
(c) Compilation of cost data on modifications	X	9				X	6				X	9				X	12				X	12			
(2) Operation and Maintenance Data																									
(a) Collection and compilation on labor assignments and costs	X	16				X	13				X	16				X	20				X	12			
(b) Collection and compilation of data on supplies and services	X	16				X	13				X	16				X	20				X	12			

TABLE G-2. (Continued)

																Combined Demonstration Program									
	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components - Alternate Plan					Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
(3) Hydrological Data																									
(a) Weekly collection of gaging station recorder charts	X	16									X	16													
(b) Weekly collection of storage reservoir recorder charts	X	16									X	16				X	20								
(c) Weekly collection of continuous rainfall and snowfall data	X	24				X	21				X	24				X	20								
(d) Monthly surveys to assess location and extent of construction	X	24				X	21				X	24				X	20								
(4) Pollution Control Data and Tests																									
(a) Weekly collection of data samples from gaging stations during periods without rainfall	X	16									X	16													
(b) Daily collection of hourly storm samples for gaging stations	X	16									X	16													
(c) Periodic sampling of reservoir and pretreatment unit influent and effluent	X	15				X	13				X	15				X	20								
(d) Weekly collection of hourly data on reservoir flows and effluent flow rates	X	15				X	13				X	15				X	20								
(e) Monthly collection of ecological samples from stream beds	X	24									X	24				X	20								
(f) Quarterly surveys of stream profiles and crossections	X	24									X	24				X	20								
(g) Special tests on pretreatment unit performance	X	6				X	13				X	6				X	6								
(h) Special test on chemical addition to pretreatment units	X	6				X	13				X	6				X	6								
(i) Tests with variable flow release rates from reservoirs	X	6									X	6				X	6								
(j) Test on interaction of reservoirs	X	6									X	6				X	6								
(5) Reuse Treatment Plant Data																									
(a) Special tests using various combinations of treatment, (i.e., chemical addition, filtration, sedimentation, carbon filtration and chlorination	X	2				X	2				X	4									X	12			
(b) Recording of treatment plant flows	X	7				X	8				X	7									X	12			
(c) Daily sampling of influent and effluent	X	7				X	8				X	5									X	12			
(d) Documentation of operating problems	X	2				X	2				X	2									X	12			
(6) Reuse System Data						X	6														X	12			
(a) Recording of quantities of water treated and supplied	X	4				X	6				X	3									X	6			
(b) Recording of makeup required	X	4				X	6				X	3									X	6			
(c) Recording of water released from reservoirs without use	X	4				X	6				X	3									X	6			
(d) Treated water consumption by users	X	4				X	6				X	3									X	6			
(e) Water consumption by comparative group of public water users	X	4				X	6																		
(f) Documentation of comments on user acceptance	X	4				X	6				X	3									X	6			

TABLE G-2. (Continued)

	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components-Alternate Plan					Combined Demonstration Program									
																Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
(7) General																									
(a) Documentation of local resident reaction and comments	X	1				X	1				X	1				X	1				X	1			
(b) Survey of residents near reservoirs	X	1				X	1				X	1				X	1				X	1			
(c) Documentation of governmental comments and approvals to the program	X	1				X	1				X	1				X	1				X	1			
Subtotal			76.2	5.6	81.8			51.8	.7	52.5			74.5	5.6	80.1			44.9	5.1	50.0			50.4	1.5	51.9
d. Supporting Laboratory Analysis		24					13					24					20								
(1) Weekly analysis of water samples collected from gaging stations	X	16				X	13				X	16													
(2) Analysis of water samples collected during storms at gaging stations	X	16									X	16													
(3) Analysis of water samples collected from reservoirs and pretreatment units	X	15				X	13				X	16				X	20								
(4) Analysis of water samples in support of test program in pretreatment unit performance tests	X	15				X	13				X	16				X	6								
(5) Analysis of water samples in support of the treatment plant testing	X	3				X	2				X	3									X	6			
(6) Analysis of water samples in connection with the operation of the treatment plant	X	4				X	6				X	3									X	12			
(7) Analysis of samples collected from stream beds	X	24									X	24				X	20								
Subtotal			46.4	1.9	48.3			13.5	.8	14.3			46.4	1.9	48.3			26.6	1.0	27.6			21.5	1.1	22.6
e. Analysis and Evaluation																									
(1) General		16					8					16													
(a) Comparison of actual and predicted design and construction costs	X	2				X	2				X	2				X	2				X	2			
(b) Evaluation of operating and maintenance costs	X	16				X	8				X	16				X	24				X	12			
(c) Determination of acceptability of using local storage reservoirs	X	16				X	8				X	16				X	2								
(d) Evaluate public and governmental acceptance of reuse concept	X	16				X	8				X	16				X	2				X	2			
(e) Determine the problems involved in the design, construction, and operation	X	16				X	8				X	16				X	2				X	2			
(2) Hydrological of local storage pollution control and storm water reuse system		16																							
(a) Verification of the hydrograph synthesis procedure for calculating runoff values from ungaged watersheds	X	16				X	8				X	16				X	9								
(b) Verification of the hydrograph systhesis procedure to determine increased runoff due to urban development	X	16				X	8				X	16				X	9								

TABLE G-2. (Continued)

	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components Alternate Plan					Combined Demonstration Program Pollution Control					Combined Demonstration Program Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
(c) Verification of the analytical methods used to calculate peak flows as a function of storm intensity	X	16				X	8				X	16				X	9								
(d) Determine effect of storage on the dampening of storm water hydrographs	X	6				X	8				X	6				X	9								
(3) Pollution																									
(a) Determine pollutant content as a function of season	X	16				X	8				X	16				X	9								
(b) Determine pollutant content as a function of storm duration	X	16				X	8				X	16				X	9								
(c) Determine pollutant content as a function of storm intensity	X	16				X	8				X	16				X	9								
(d) Determine pollutant content as a function of urbanization	X	16				X	8				X	16				X	9								
(e) Determine velocity - pollutant content relationship for storm flows	X	16				X	8				X	16				X	9								
(4) Pollution Control																									
(a) Determine adequacy of individual reservoirs to collect and treat storm flow	X	16				X	8				X	16				X	9								
(b) Determine performance characteristics of tube settlers for treating storm water	X	16				X	8				X	16				X	9								
(c) Determine effect of chemical addition on performance characteristics of tube settlers	X	16				X	8				X	16				X	9								
(d) Determine the effect of dampening of storm water flow on stream erosion	X	6									X	5				X	8								
(e) Evaluate the interaction of multiple reservoir installations	X	6									X	6				X	6								
(f) Establish methods for the design and analysis of multiple reservoir installation	X	6									X	6				X	6								
(g) Compare the overall effectiveness of local storage pollution control with uncontrolled areas	X	6									X	6				X	6								

TABLE G-2. (Continued)

	Demonstration of the Overall System					Demonstration of System Components					Demonstration of System Components - Alternate Plan					Combined Demonstration Program									
																Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
(5) Treatment																									
(a) Evaluate the effectiveness of various processes for the treatment of storm water	X	7				X	4				X										X	12			
(b) Establish operating and maintenance costs for various treatment methods	X	4				X	6				X	5									X	12			
(c) Determine adequacy of storm water supplies for meeting the water demands	X	4				X	6				X	3									X	6			
(d) Determine overall treatment costs as a function of quantity and quality	X	4				X	6				X	5									X	6			
(6) Reuse																									
(a) Evaluate the acceptability of storm water reuse	X	4				X	6				X	3									X	6			
(b) Determine quantities that could be supplied by reuse as a function of quality	X	4				X	6				X	3									X	6			
(c) Compare internal usage of water and without reuse	X	4				X	6																		
(d) Evaluate the potential of reuse as a supplemental water supply	X	4				X	6				X	3									X	6			
(e) Evaluate methods for distributing treated water for reuse	X	4				X	6																		
Subtotal			52.9	2.8	55.7			30.6	1.5	32.1			49.7	2.8	52.5			31.6	2.8	34.4			13.9	1.4	15.3
f. System Application and Development		16										16					2								
(1) Review actual results with analytical predictions	X	4				X	5				X	4				X	2								
(2) Modify analytical methods based on actual test data	X	4				X	5				X	4				X	2								
(3) Develop design guidelines and criteria for the application of the system for various conditions of rainfall, runoff, demand, and water cost	X	4				X	5				X	4				X	2								
(4) Prepare data on the probable costs of constructing and operating system	X	4				X	5				X	4				X	2								
(5) Provide data on the effectiveness of the system for pollution control	X	4									X	4				X	2								
(6) Provide data on the effectiveness of the system as a supplemental source of water	X	4									X	4													
Subtotal			25.4	2.3	27.7			13.6	.8	14.4			25.4	2.3	27.7			17.0	1.6	18.6			10.8	.9	11.7
g. Reports																									
(1) Prepare monthly status reports	X	22				X	19				X	22				X	30				X	24			
(2) Prepare quarterly technical progress reports	X	22									X	22				X	30				X	24			
(3) Prepare final reports for each phase	X	2				X	2				X	2				X	2				X	2			
Subtotal			28.9	.6	29.5			13.0	.4	13.4			28.9	.6	29.5			22.5	.6	23.1			27.2	.5	27.7
3. Total Test and Evaluation			321.4	56.1	377.5			192.6	16.3	208.9			316.5	48.1	364.6			254.7	33.2	287.9			222.0	33.4	255.4

TABLE G-3. APPLICATION PROJECTS - OPERATION AND MAINTENANCE COSTS

(Costs in Thousands)

	Combined Demonstration Program									
	Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
Operation and Maintenance										
a. Storage Reservoirs and Pretreatment Systems	11	15								
(1) Weekly check and replenishment of chemicals	X	15								
(2) Weekly cleaning of Tube Settler Basins	X	15								
(3) Monthly preventative maintenance	X	15								
(4) Quarterly cleaning of reservoirs	X	15								
(5) Algae and pest control	X	15								
(6) General Maintenance	X	15								
Subtotal			14.7	14.9	29.6					
b. Treatment Plants and Pumping Facilities						9	6			
(1) Checkout and initial operation						X	3			
(2) Daily equipment checks, sample collection, and chemical replenishment						X	6			
(3) Monthly preventative maintenance						X	6			
(4) Repairs						X	6			
(5) Electrical Energy						X	6			
Subtotal								40.2	6.5	46.7
Totals			14.7	14.9	29.6			40.2	6.5	46.7

TABLE G-4. APPLICATION PROJECTS - TEST AND EVALUATION COSTS
(Costs in Thousands)

	Combined Demonstration Program									
	Pollution Control					Storm Water Reuse				
	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost	No. of Units/Applicability	Performance Period (Months)	Labor Cost	Material Cost	Total Cost
Test and Evaluation										
a. Planning, Management and Coordination	X	30				X	18			
(1) Develop design criteria guidelines, and specifications	X	1				X	3			
(2) Select design contractor	X	1				X	1			
(3) Review and approval of drawings and specifications	X	4				X	6			
(4) Preparation of bid documents	X	1				X	1			
(5) Bid evaluation and contract negotiation	X	1				X	1			
(6) Monitoring of construction and field inspection	X	6				X	9			
(7) Review and analysis of test results	X	15				X	6			
(8) Direction of report preparation	X	4				X	3			
Subtotal			60.9	1.4	62.3			36.3	.5	36.8
b. Initial Test and Evaluation		15					18			
(1) Test Program Planning	X	12				X	12			
(2) Testing and Data Collection	X	3				X	3			
(3) Supporting Laboratory Analysis	X	3				X	3			
(4) Analysis and Evaluation of test data	X	3				X	3			
(5) System modifications	X	3				X	3			
(6) Report Preparation	X	3				X	3			
Subtotal			50.7	15.5	66.2			69.0	10.5	79.5
c. Overall Test and Evaluation										
(1) Data Collection	X	15				X	3			
(2) Supporting Laboratory Analysis	X	15				X	3			
(3) Analysis and Evaluation of System Operation	X	15				X	3			
(4) Analysis and Evaluation of System Effectiveness	X	9				X	3			
(5) System Application Evaluation	X	6				X	3			
(6) Report Preparation	X	3				X	2			
Subtotal			70.8	1.4	72.2			35.4	.9	36.3
d. Totals			182.4	18.3	200.7			140.7	11.9	152.6

APPENDIX H

PACKAGED WATER TREATMENT PLANTS

As stated in Chapter IV and Appendix F, the treatment plant shown in the Demonstration System Preliminary Design Report, Appendix F, would not be typical of the treatment plants used in future local storage, treatment, and reuse systems. After the processes for treatment of storm water have been evaluated, it is envisioned that future treatment plants specifically for storm water will be manufactured.

This appendix contains drawings of a presently manufactured packaged treatment plant. This particular plant is rated at a 20 gpm nominal capacity and utilizes coagulating chemicals and provides flocculation, sedimentation, and mixed media filtration in a single, compact, factory built unit.

These plants have been satisfactorily operated with minimal operator attendance. The flow into the unit is manually set by use of a plug valve. The flow out of the unit is automatically controlled to match that entering by a float-controlled value located in the filter effluent line. A motor-driven flocculator is employed to form a settleable floc, and a tube settler is utilized for efficient removal of the settleable floc previous to mixed-media filtration. Backwash of the unit is automatically initiated upon high headloss across the filter or may be manually initiated by the plant operator. During the backwash cycle, a siphon is employed to remove the water and sludge from the unit. Rapid drawdown of the water level provides efficient flushing of the settling tubes. The chemical feed pumps automatically start and stop as required when the plant goes into backwash and returns to normal operation. The pump located on the plant influent line will shut off when the unit goes into backwash and start upon returning to normal operation.

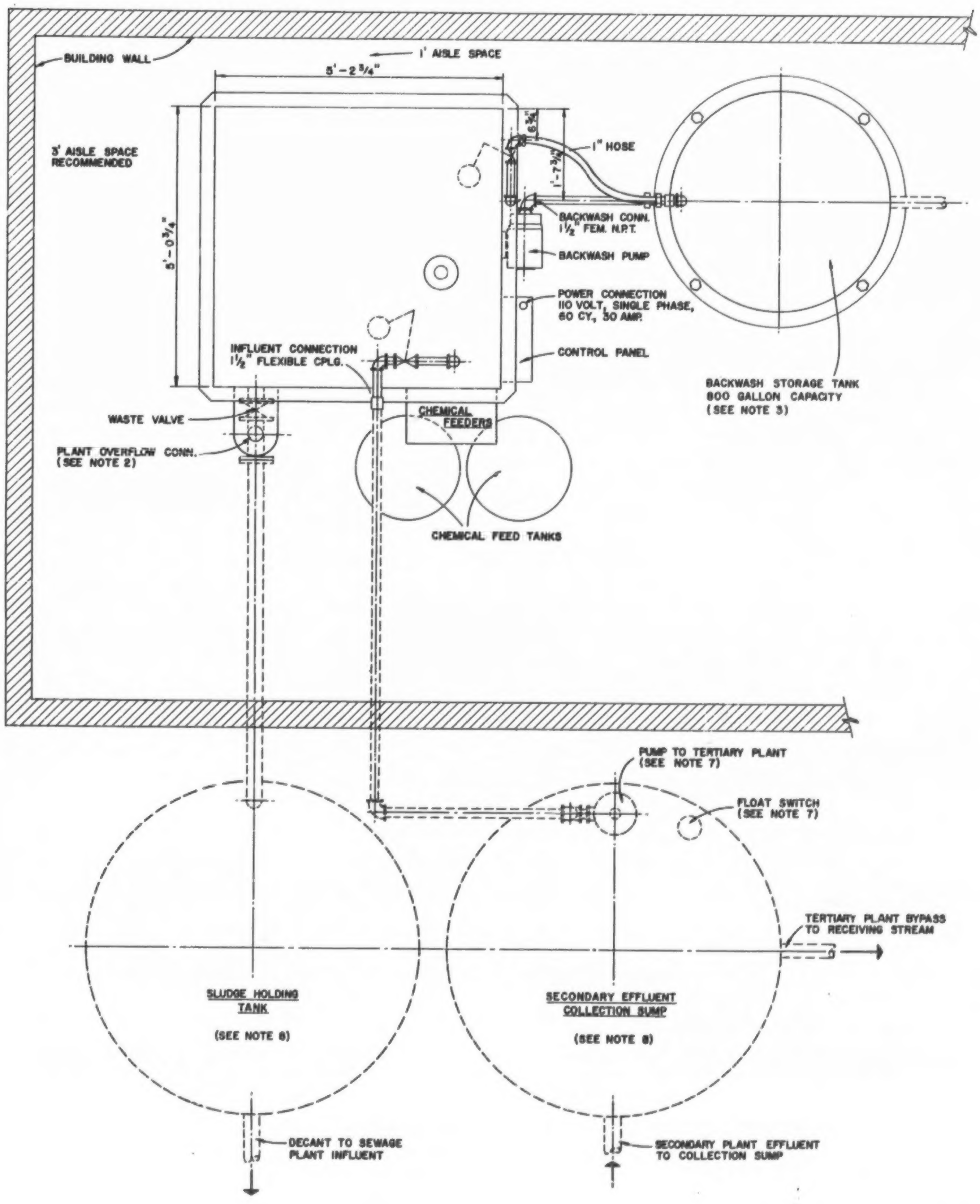
Equipment is available for indicating and recording the plant effluent turbidity and this equipment can be adjusted to automatically shutdown the plant at any turbidity level between 0 to 10 Jackson Units. In addition, equipment is also available automatically to transmit performance and operating data over telephone lines to a central location to permit the operation of a number of plants to be controlled by a single operator.

The price of a 20 gpm treatment plant of this type is less than $14,000 without the special monitoring equipment. The following figures are included in this Appendix:

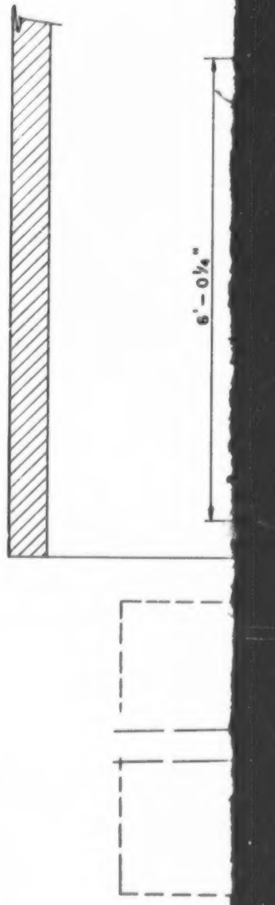
(1) Figure H-1 shows a typical installation.

(2) Figure H-2 shows details of a typical unit.

(3) Figure H-3 is a flow diagram for the unit shown on Figure H-2.


BUILDING WALL
1' AISLE SPACE
5'-2 3/4"
3' AISLE SPACE RECOMMENDED
6 3/4"
1"-7 3/4"
1" HOSE
5'-0 3/4"
BACKWASH CONN. 1 1/2" FEM. N.P.T.
BACKWASH PUMP
POWER CONNECTION 110 VOLT, SINGLE PHASE, 60 CY., 30 AMP.
INFLUENT CONNECTION 1 1/2" FLEXIBLE CPLG.
CONTROL PANEL
BACKWASH STORAGE TANK 800 GALLON CAPACITY (SEE NOTE 3)
WASTE VALVE
CHEMICAL FEEDERS
PLANT OVERFLOW CONN. (SEE NOTE 2)
CHEMICAL FEED TANKS
PUMP TO TERTIARY PLANT (SEE NOTE 7)
FLOAT SWITCH (SEE NOTE 7)
TERTIARY PLANT BYPASS TO RECEIVING STREAM
SLUDGE HOLDING TANK
(SEE NOTE 8)
SECONDARY EFFLUENT COLLECTION SUMP
(SEE NOTE 8)
DECANT TO SEWAGE PLANT INFLUENT
SECONDARY PLANT EFFLUENT TO COLLECTION SUMP

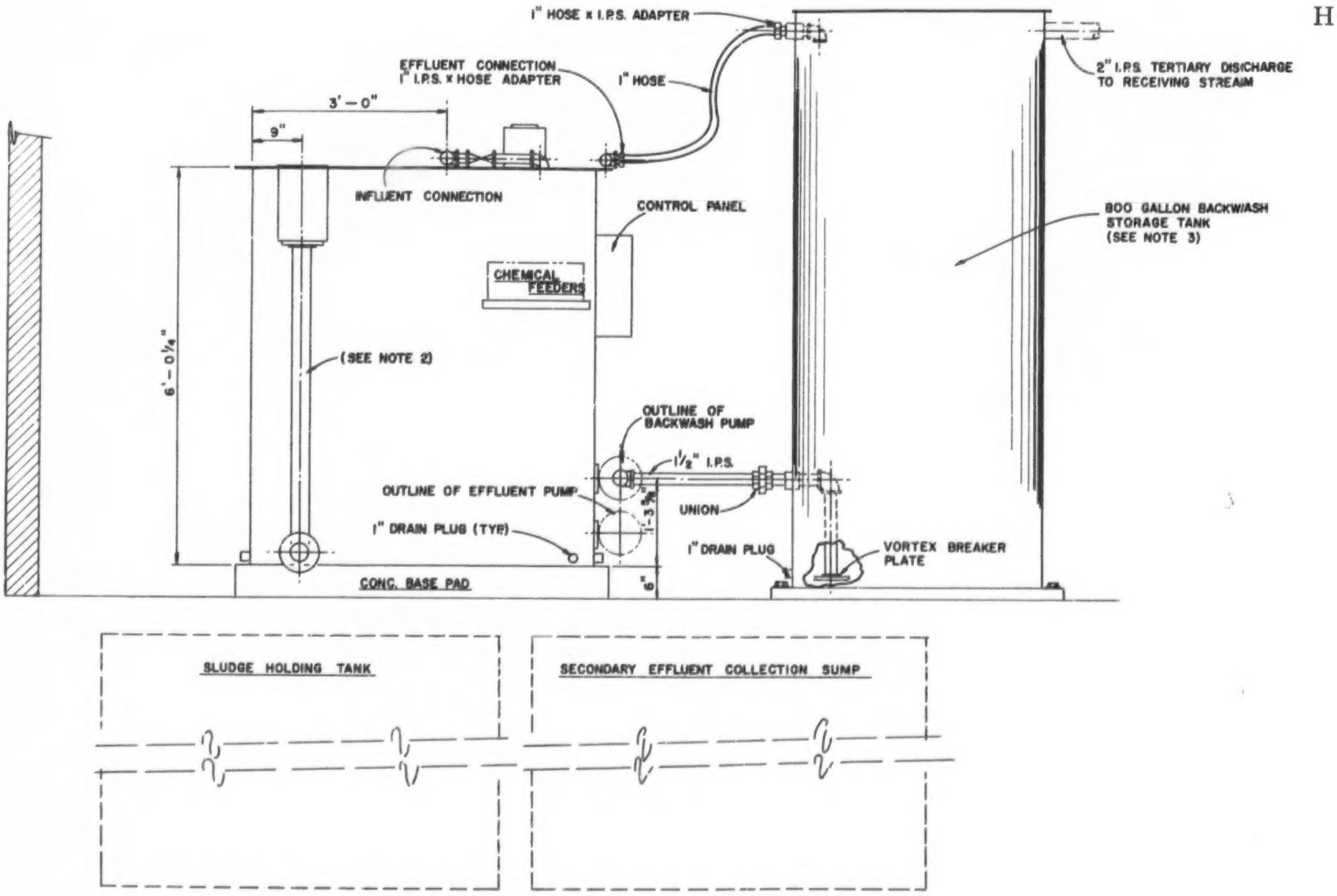
PLAN LAYOUT



NOTES:

1. DETAI
2. OVERF
3. BACKW
THE B
4. CHEMI
INSTAL
5. FILTE
6. FLOOR
7. PUMP
PANEL
8. TOP O

A


1" HOSE x I.P.S. ADAPTER
EFFLUENT CONNECTION
1" I.P.S. x HOSE ADAPTER
1" HOSE
2" I.P.S. TERTIARY DISCHARGE TO RECEIVING STREAM
3'-0"
9"
INFLUENT CONNECTION
CONTROL PANEL
800 GALLON BACKWASH STORAGE TANK (SEE NOTE 3)
CHEMICAL FEEDERS
6'-0¼"
(SEE NOTE 2)
OUTLINE OF BACKWASH PUMP
1½" I.P.S.
OUTLINE OF EFFLUENT PUMP
1'-3⅝"
UNION
1" DRAIN PLUG (TYP)
1" DRAIN PLUG
VORTEX BREAKER PLATE
CONC. BASE PAD
6"
SLUDGE HOLDING TANK
SECONDARY EFFLUENT COLLECTION SUMP

ELEVATION

NOTES:

1. DETAILS SUBJECT TO CHANGE UNLESS THIS DRAWING MARKED "APPROVED FOR CONSTRUCTION".
2. OVERFLOW PIPE SHIPPED LOOSE FOR FIELD ASSEMBLY.
3. BACKWASH PIPE, UNION, EFFLUENT HOSE AND FITTINGS ARE SHIPPED LOOSE AS PART OF THE BACKWASH TANK FOR FIELD INSTALLATION.
4. CHEMICAL FEED TANKS AND FEEDERS' SUCTION HOSES ARE SHIPPED LOOSE FOR FIELD INSTALLATION.
5. FILTER MEDIA SHIPPED IN BAGS FOR FIELD INSTALLATION.
6. FLOOR DRAIN RECOMMENDED IN VICINITY OF CHEMICAL FEED TANKS.
7. PUMP SEQUENCED BY FLOAT SWITCH IN SUMP. POWER AND CONTROL FROM RECLAMATE PANEL. THREE #12 WIRES TO CONTROL PANEL REQUIRED – FIELD INSTALLED.
8. TOP OF TANKS ASSUMED AT GRADE ELEVATION. SPECIAL ARRANGEMENTS CAN BE MADE.

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TYPICAL INSTALLATION

RECLA-MATE
MODEL SWB-27
TERTIARY TREATMENT PLANT

SCALE	NONE	DATE	DRAWING NO.
DRAWN	J.K.		M- 374
CHECKED			
APPROVED			SHEET 1 OF 1

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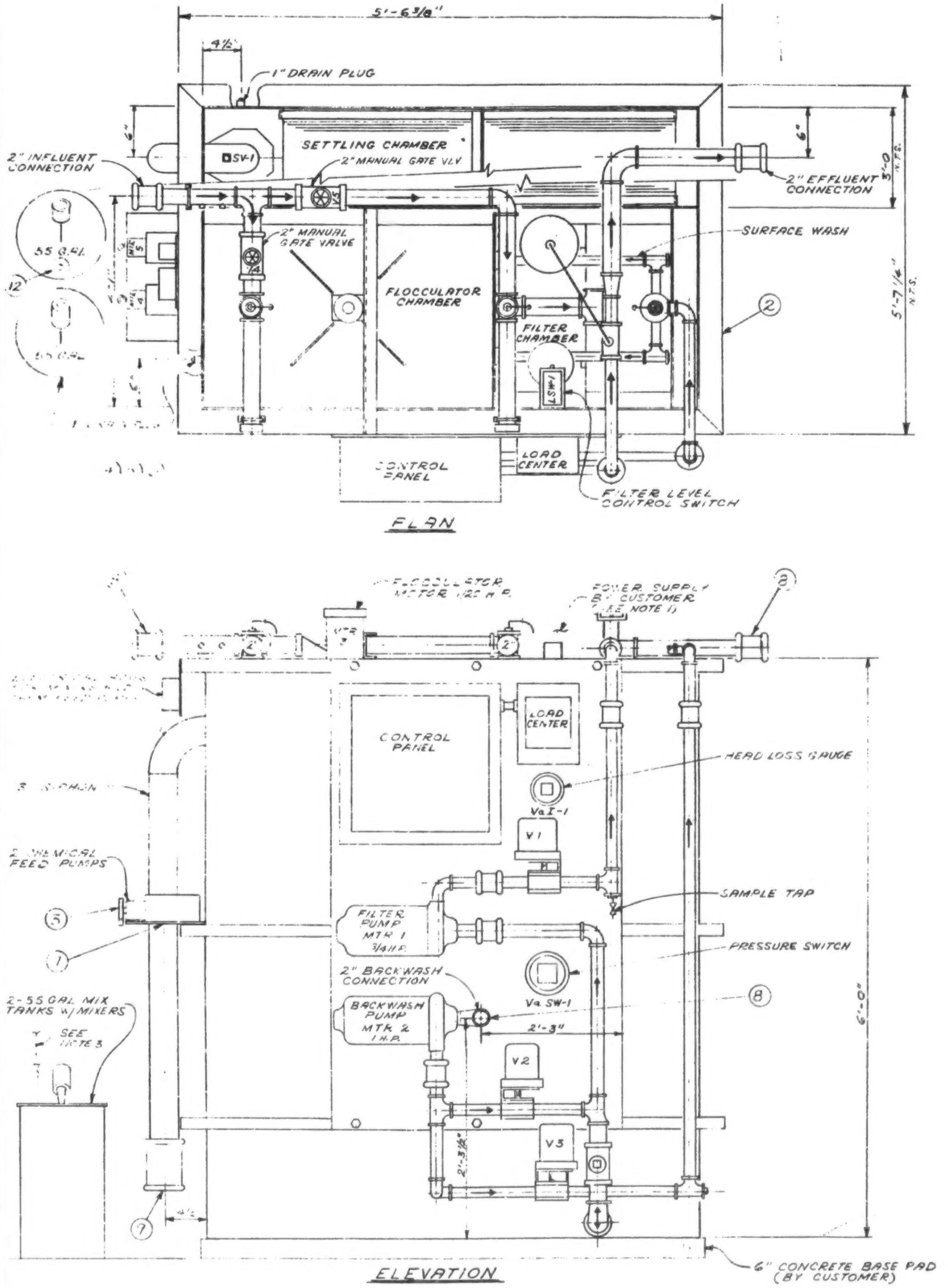
Figure H-1

B


5'-6 3/8"
4½"
1" DRAIN PLUG
6"
SETTLING CHAMBER
SV-1
2" INFLUENT CONNECTION
2" MANUAL GATE VLV
2" EFFLUENT CONNECTION
3'-0" N.T.S.
2" MANUAL GATE VALVE
55 GAL
12
55 GAL
FLOCCULATOR CHAMBER
FILTER CHAMBER
SURFACE WASH
2
5'-7¼" N.T.S.
LSW-1
CONTROL PANEL
LOAD CENTER
FILTER LEVEL CONTROL SWITCH
FLAN
FLOCCULATOR MOTOR 1/20 H.P.
POWER SUPPLY BY CUSTOMER (SEE NOTE 1)
8
CONTROL PANEL
LOAD CENTER
HEAD LOSS GAUGE
Va I-1
V1
3" SIPHON
2 CHEMICAL FEED PUMPS
5
7
SAMPLE TAP
FILTER PUMP MTR 1 3/4 H.P.
PRESSURE SWITCH
2" BACKWASH CONNECTION
Va SW-1
8
BACKWASH PUMP MTR 2 1 H.P.
2'-3"
2-55 GAL MIX TANKS W/ MIXERS
SEE NOTE 3
V2
V3
2'-3½"
6'-0"
4½
7
6" CONCRETE BASE PAD (BY CUSTOMER)
ELEVATION

SWB-27 ARRANGEMENT ①

EQUIPMENT SUPP

QTY	ITEM	DESCR
X	1	SWB-27 ARRAN
1	2	SWB-27 FINAL
2	3	CHEMICAL FEED
2	4	CHEMICAL MIX TAN
2	5	MIXER 115V
2	6	MIXER PEELER
1	7	FEEDER SUPP
3	8	FLEXIBLE CO
1	9	FLEXIBLE COUPLIN
1	10	PARTS KIT
LOT	11	FILTER MEDIA
1	12	DISPERSER

A

NOTES:

1. POWER SUPPLY TO BE 120/240 VOLTS, 3 WIRE, SINGLE PHASE, 60 CYCLE, 30 AMP.
2. FOR DIRECT FILTERATION BYPASSING FLOCCULATION & SETTLING CLOSE V4 & OPEN V5.
3. WATER LINE AND HOSE RECOMMENDED FOR CHEMICAL MIXING. USE PRESSURIZED FINISHED WATER.
4. PARTS KIT ITEM 10, INCLUDES MISC. HARDWARE, SPARE PARTS & OPERATING AIDS.

EQUIPMENT SUPPLIED BY NEPTUNE MICROFLOC

QTY	ITEM	DESCRIPTION	REFERENCE
☒	1	SWB-27 ARRANGEMENT	
1	2	SWB-27 FINAL ASSEMBLY	P4156-5
2	3	CHEMICAL FEED PUMP - 36 GPD CHEMCON RA DEN 35	
2	4	CHEMICAL MIX TANK - 55 GAL. DELAWARE BARREL	NO. [illegible]
2	5	MIXER 115V [illegible], 3 SPEED w/316SS SHAFT & PROP.	LIGHTNIN NO. [illegible]
2	6	MIXER & [illegible] FEEDER BRACKET ASSY. 55GAL.	E110-300 ITEM 2
1	7	FEEDER SUPPORT	E091-00[illegible] IT. 1
3	8	FLEXIBLE COUPLING 2" - AERO-QUIP NO. 6600-[illegible]-400	
1	9	FLEXIBLE COUPLING 3" - AERO-QUIP NO. 6600-3-650	
1	10	PARTS KIT	
LOT	11	FILTER MEDIA	
1	12	DISPERSER DOW CHEMICAL	

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SNB-27 ARRANGEMENT

SCALE	NONE	DATE	DRAWING NO.
DRAWN	F. Bruno	19 JULY 68	P4156-1
CHECKED			
APPROVED	Buckley	22 Jul 68	SHEET 1 OF 1

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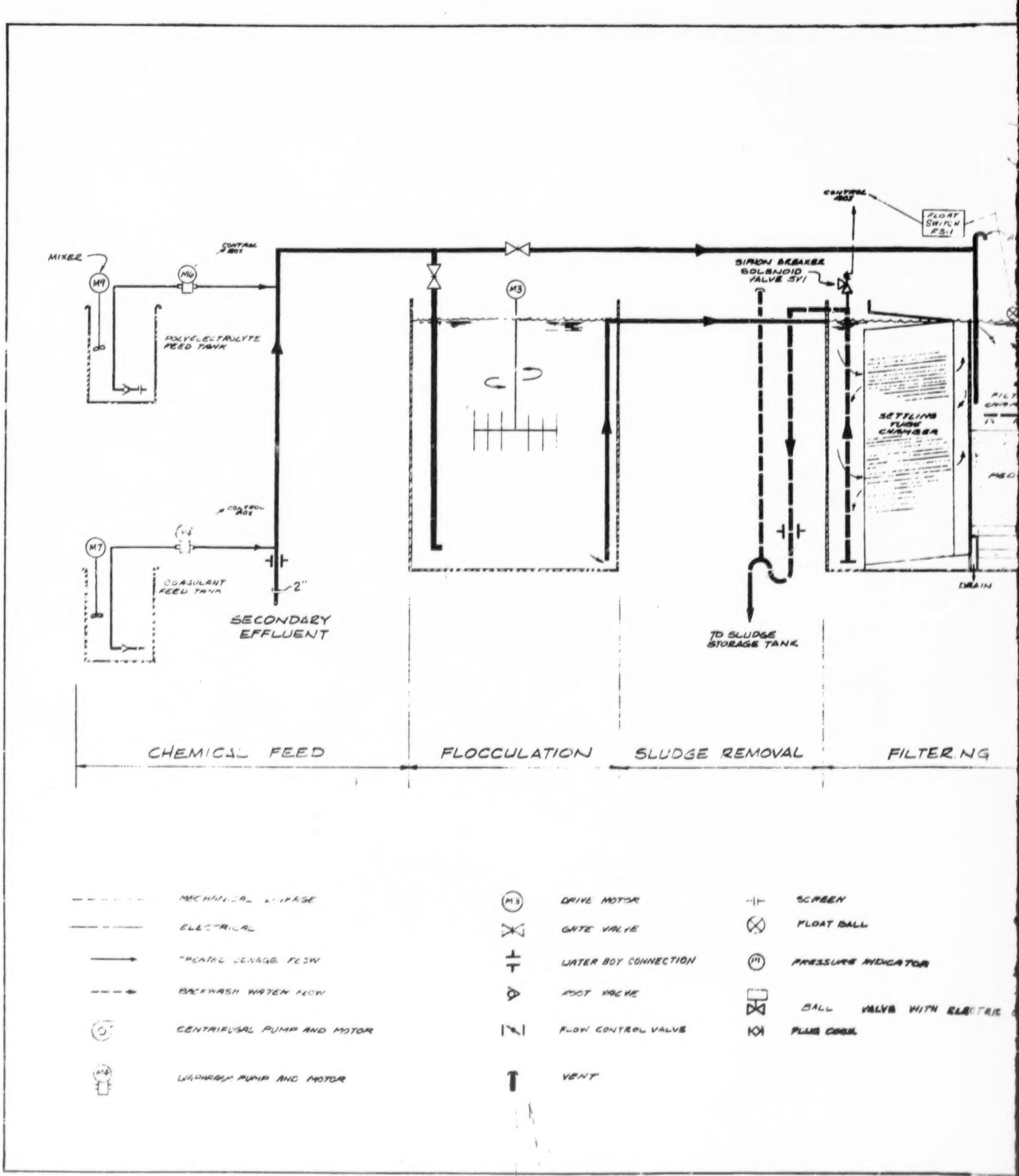
CORVALLIS, OREGON

Figure H-2

B


MIXER
M9
M6
CONTROL BOX
POLYELECTROLYTE FEED TANK
M7
M8
CONTROL BOX
COAGULANT FEED TANK
2"
SECONDARY EFFLUENT
M3
SIPHON BREAKER SOLENOID VALVE SV1
CONTROL BOX
FLOAT SWITCH FS-1
SETTLING TUBE CHAMBER
DRAIN
TO SLUDGE STORAGE TANK
CHEMICAL FEED
FLOCCULATION
SLUDGE REMOVAL
FILTERING
MECHANICAL LINKAGE
ELECTRICAL
TREATED SEWAGE FLOW
BACKWASH WATER FLOW
CENTRIFUGAL PUMP AND MOTOR
M8
DIAPHRAGM PUMP AND MOTOR
M3
DRIVE MOTOR
GATE VALVE
WATER BOY CONNECTION
FOOT VALVE
FLOW CONTROL VALVE
VENT
SCREEN
FLOAT BALL
PI
PRESSURE INDICATOR
BALL VALVE WITH ELECTRIC
PLUG COCK

A

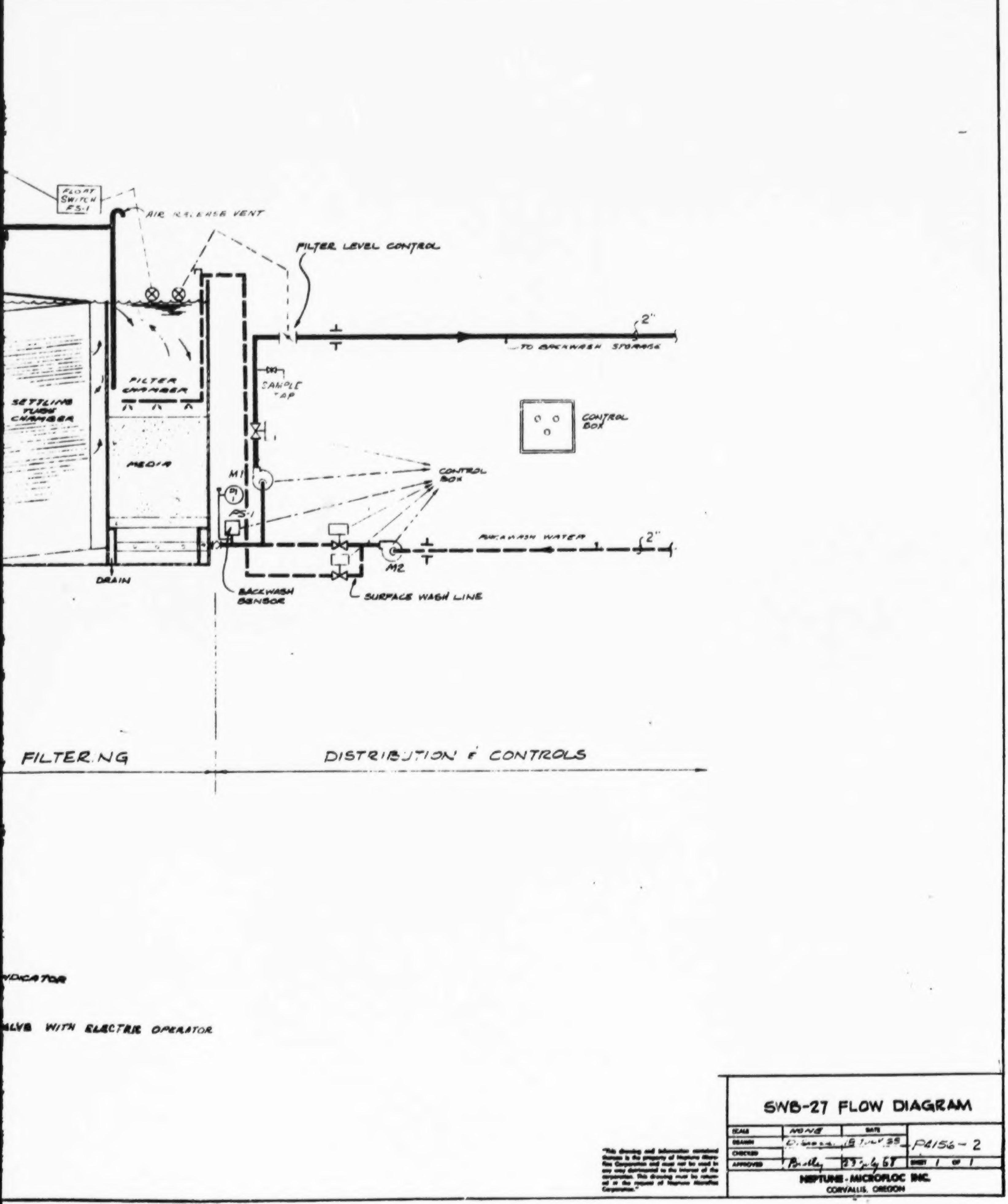

FLOAT SWITCH FS-1
AIR RELEASE VENT
FILTER LEVEL CONTROL
2"
TO BACKWASH STORAGE
SAMPLE TAP
FILTER CHAMBER
SETTLING TUBE CHAMBER
MEDIA
CONTROL BOX
CONTROL BOX
M1
PI 1
PS-1
BACKWASH WATER
2"
M2
DRAIN
BACKWASH SENSOR
SURFACE WASH LINE
FILTERING
DISTRIBUTION & CONTROLS
NDICATOR
ALVE WITH ELECTRIC OPERATOR
SWB-27 FLOW DIAGRAM
SCALE
NONE
DATE
DRAWN
15 JULY 55
P4156 - 2
CHECKED
APPROVED
Bradley
23 July 68
SHEET 1 OF 1
NEPTUNE-MICROFLOC INC.
CORVALLIS, OREGON

Figure II-3

B